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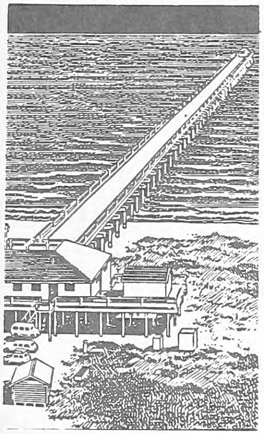
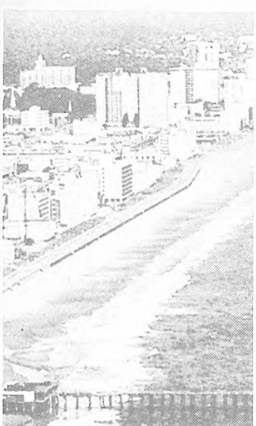




TECHNICAL REPORT CERC-88-1

COASTAL ENGINEERING STUDIES IN SUPPORT OF VIRGINIA BEACH, VIRGINIA, BEACH EROSION CONTROL AND HURRICANE PROTECTION PROJECT

Report 1

PHYSICAL MODEL TESTS OF IRREGULAR WAVE OVERTOPPING AND PRESSURE MEASUREMENTS

by

Martha S. Heimbaugh, Peter J. Grace, John P. Ahrens
D. Donald Davidson

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

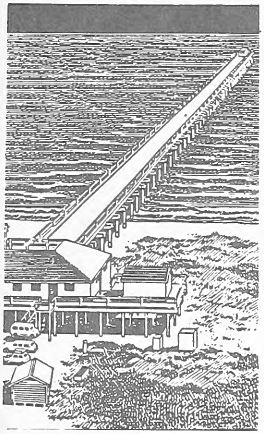
March 1988
Report 1 of a Series

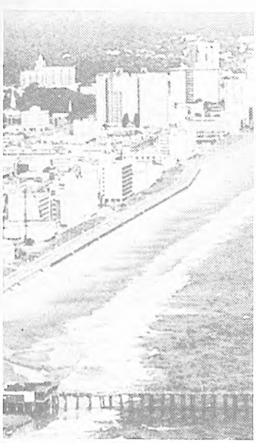
Approved For Public Release; Distribution Unlimited

Prepared for US Army Engineer District, Norfolk
Norfolk, Virginia 23510-1096

Under Intra-Army Order No. AD-86-3018

US Army Corps of Engineers





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19. ABSTRACT *(Continue on reverse if necessary and identify by block number)*

A two-dimensional (2-D) physical model investigation was conducted at scales of 1:13 and 1:19 (model to prototype) to provide input for the design optimization of a seawall proposed for long-term storm protection at Virginia Beach, Virginia. This was one of a number of tasks conducted in support of the detailed design of a beach erosion control and hurricane protection project at Virginia Beach. The 2-D tests were conducted to acquire data on the expected rate of overtopping for two design storm types (hurricane and northeaster) at selected still-water levels, determine a stable stone size for a proposed fronting riprap berm, and to determine the distribution of wave-induced pressures on the face of the seawall.

As a result of the 2-D tests, a stable stone size was determined for the proposed fronting berm, and overtopping rates were measured. An improved seawall design was

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recommended and showed a significant reduction of overtopping rates over the initial seawall design. Wave-induced shock pressures were recorded on the face of the seawall; however, durations were small and probably insignificant. Measured surge pressure magnitudes were relatively consistent and durations were significant. No significant negative pressures were recorded.

PREFACE

The US Army Engineer District, Norfolk (CENAO), requested the US Army Engineer Waterways Experiment Station's (CEWES's) Coastal Engineering Research Center (CERC) to conduct physical model studies to determine overtopping rates and wave-induced pressures on a seawall proposed for construction at Virginia Beach, Virginia. This is the first of three reports that describe tasks conducted in support of the Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project. Funding authorizations by CENAO were granted in accordance with Intra-Army Order No. AD-86-3018.

Physical model tests were conducted at CERC under general direction of Dr. James R. Houston, Chief, CERC; Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC; Mr. C. Eugene Chatham, Chief, Wave Dynamics Division; and Mr. D. Donald Davidson, Wave Research Branch (CW-R). Tests were conducted by Messrs. Cornelius Lewis, Sr., Engineering Technician, John M. Heggins, Computer Technician, and Lonnie L. Friar, Electronics Technician, under the supervision of Ms. Martha S. Heimbaugh, Civil Engineer, and Mr. P. J. Grace, Hydraulic Engineer, CW-R. Mr. Kenneth W. Hassenflug, Computer Specialist, CW-R, was responsible for software development throughout execution of the pressure tests and during subsequent data analysis efforts. This report was prepared by Ms. Heimbaugh and Messrs. Grace, Davidson, and John P. Ahrens, Oceanographer, CW-R. Report editing was performed by Ms. Shirley A. J. Hanshaw, Information Products Division, Information Technology Laboratory, CEWES.

Throughout the course of this study liaison was maintained with Ms. Joan Pope, CERC's overall Project Manager, and CENAO representatives: Messrs. David Pezza, Project Manager, Owen Reece, Hydraulic Engineer, and Steve Geusik, Structural Engineer. The contributions of these individuals, and all other involved CENAO personnel, are acknowledged with thanks for their assistance in the investigation.

Commander and Director of CEWES during the investigation and the preparation and publication of this report was COL Dwayne G. Lee, CE. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet per second per foot	0.09290	cubic metres per second per foot
feet	0.3048	metres
inches	2.540	centimetres
miles	1.6093	kilometres
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	cubic metre

COASTAL ENGINEERING STUDIES IN SUPPORT OF VIRGINIA BEACH, VIRGINIA BEACH EROSION CONTROL AND HURRICANE PROTECTION PROJECT

Report 1

Physical Model Tests of Irregular Wave Overtopping and Pressure Measurements

PART I: INTRODUCTION

Study Background

1. This report is the first of a series of three reports on coastal engineering studies conducted by the US Army Engineer Waterways Experiment Station's (CEWES's) Coastal Engineering Research Center (CERC) to assist the US Army Engineer District, Norfolk (CENAO), in the Advanced Engineering and Design of the Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project. The other two reports concern overtopping hydrograph design and beach and dune design. The coastal studies were divided into two major sections: seawall design (i.e., the physical model overtopping and wave-induced pressure measurements and analysis of overtopping for design events) and beach and dune design evaluation (i.e., numerical simulation of profile response to short-term design events and design of beach fill for long-term stability and maintenance). Figure 1 presents a flowchart of the coastal engineering studies.

2. Selection of design waves, storm surge hydrographs, and runup-overtopping rates was crucial to development of the most hydraulically efficient seawall geometry and definition of short-term beach stability. Coastal engineering studies consisted of selecting design storms from the historical record, simulating the wave field for each of these storms, establishing design surge hydrographs, developing a two-dimensional (2-D) hydrographic model to measure overtopping rates and test wave-induced pressure loadings, computing an overtopping hydrograph adjusted for all prototype parameters, numerically simulating beach and dune response to design events, developing a design and construction beach profile for long-term adjustment, and establishing a beach maintenance plan.

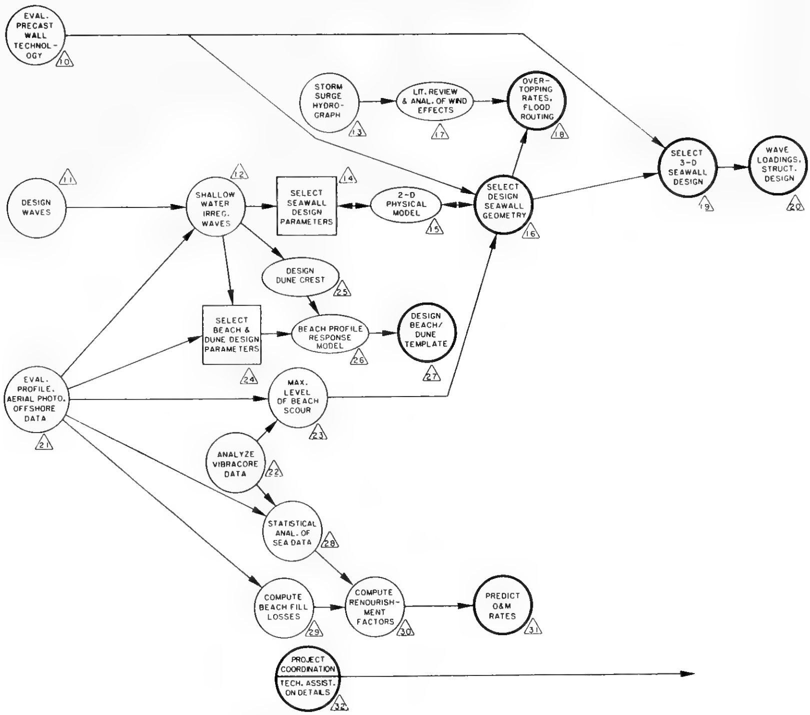

Figure 1. Flowchart for coastal engineering studies, Virginia Beach, Virginia

Site Background

3. The proposed Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project is one of the largest and most complex coastal projects of this type in recent Corps of Engineers experience. The City of Virginia Beach is located on the east coast of the United States just south of the entrance to Chesapeake Bay (Figure 2). The project area consists of 6 miles* of heavily developed commercial and urban shoreline which extends north from Rudee Inlet to 89th Street (Figure 3**). This shoreline is subject

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations (el) cited herein are in feet as referenced to National Geodetic Vertical Datum (NGVD).

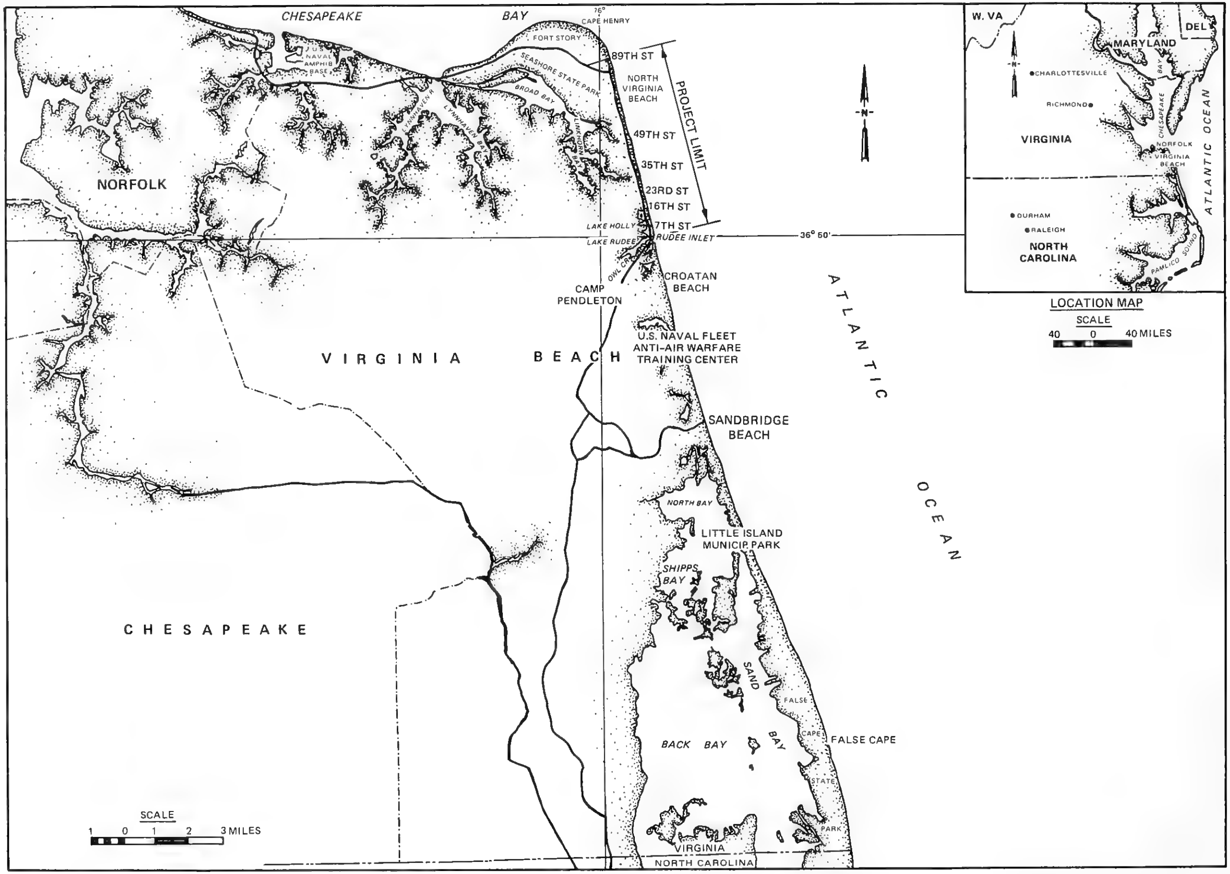

Figure 2. Location map of study site

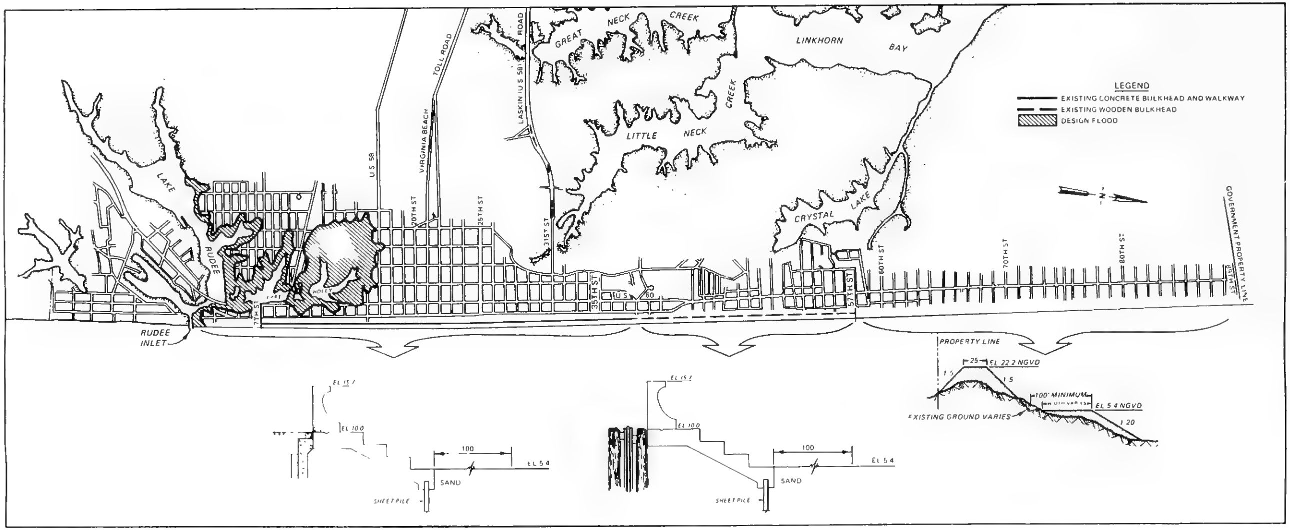

Figure 3. Project reach, Rudee Inlet to 89th Street

to severe damages from hurricanes and extreme extratropical storms (locally called northeasters). The August 1933 hurricane and the March 1962 extratropical storm (the Ash Wednesday storm) devastated this coastal area. Storm damages included loss of the beach, destruction of the bulkhead and seawall system, damage to buildings, and inshore flooding. In addition, there has been a continuing problem with beach erosion. Since 1962 annual harbor dredging of Rudee Inlet and pumping operations to bypass the sand at Rudee Inlet, and/or the trucking in of sand from other sources, has been sponsored by the Federal, state, and city governments to maintain a beach width of approximately 100 ft and a crest el of +5.4 ft.

4. Existing protection consists of a combination of various bulkheads with crest els between 10 and 12 ft NGVD and nourished beach. In 1970 CENAO completed a feasibility study which recommended construction of a sheet-pile seawall with a concrete cap at el 15 and heavy stone at the base. By 1983, results of the previous study had been reevaluated and incorporated into an initial (Phase I) seawall design and beach erosion control concept. The seawall was designed with guidance from the Shore Protection Manual (SPM) (1984) which is based primarily on monochromatic wave theory. Adequate storm protection was to be provided by the seawall without sacrificing aesthetics of the ocean view.

5. The proposed project seawall has a crest el of 15.7 ft NGVD and will extend from Rudee Inlet north to 57th Street. Beyond this point, a dune and beach system will occupy the area from 57th Street north to 89th Street. The recommended plan also calls for a 100-ft wide berm at el +5.4 ft NGVD from Rudee Inlet to 89th Street (Figure 3). When built, the seawall project should provide 54-year flood protection to the community (CENAO 1984).

Purpose of the Model Study

6. This model study was conducted to determine the adequacy of the proposed seawall design and, if necessary, to investigate the effectiveness of design modifications. The physical model study was one of a series of tasks conducted by CERC to aid in the design of the detailed Beach Erosion Control and Hurricane Protection Project for Virginia Beach. The specific purposes of this 2-D physical model study were to:

_a_. Determine the expected rate of overtopping for two design storm types (hurricane and northeaster) at four selected still-water levels (swl's).

_b_. Recommend any changes in the geometry of the seawall which might decrease the overtopping rate.

_c_. Determine a stable stone size for the proposed fronting riprap.

_d_. Evaluate the distribution of wave-induced pressures on the face of the seawall to aid in final design of the wall and foundation.

PART II: THE MODEL

Scale Selection

7. During this model study, time constraints dictated that construction of the physical model be carried out prior to determination of ultimate test conditions (by CERC's Coastal Oceanography Branch). Under these conditions a model to prototype scale of 1:13 was chosen based on calculations indicating that any smaller scale would introduce scale effects into the secondary task of optimizing a stable fronting riprap design; therefore, Phase I seawall overtopping tests were performed at the 1:13 scale. However, after the design test conditions were eventually chosen, it was found that at a scale of 1:13 only 60 percent of the design deepwater zero moment wave height H_{mo} * could be consistently achieved at the wave board for all representative swl's. At this time it was believed that overtopping was controlled by the inshore conditions. Because visual and measured observations indicated maximum wave heights and H_{mo} values were being maintained, tests were continued at this scale. After changes in the geometry of the seawall had been recommended, however, the decision was made to implement a smaller model scale of 1:19 to achieve 100 percent of the design H_{mo} at the wave board in deep water. Also at this point, a stable riprap size had been determined; therefore, all Phase II testing was performed at a model scale of 1:19. (In this report, Phase II implies that geometric modifications to the seawall had been incorporated.) Pressure tests conducted for the Phase II seawall were conducted at the 1:19 scale.

Equipment and Facilities

8. All tests were conducted in a concrete wave flume 11 ft wide and 250 ft long. The cross section of the tank in the vicinity of the structure was partitioned into two 3-ft-wide channels and two 2.5-ft-wide wave-absorbing channels (Figure 4). Irregular waves were generated by a hydraulically actuated piston-driven wave board. The seawall test sections were installed in the flume approximately 200 ft from the wave board.

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* For convenience, symbols and abbreviations are listed in the Notation (Appendix E).

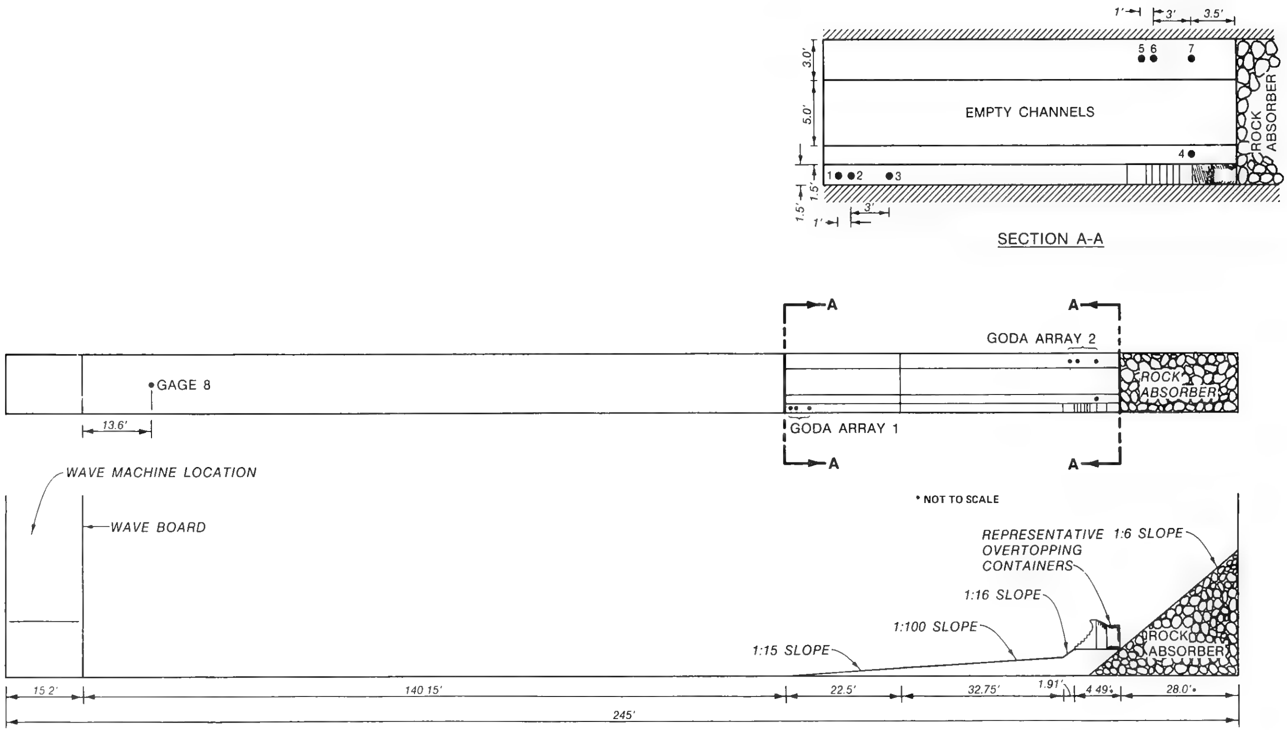


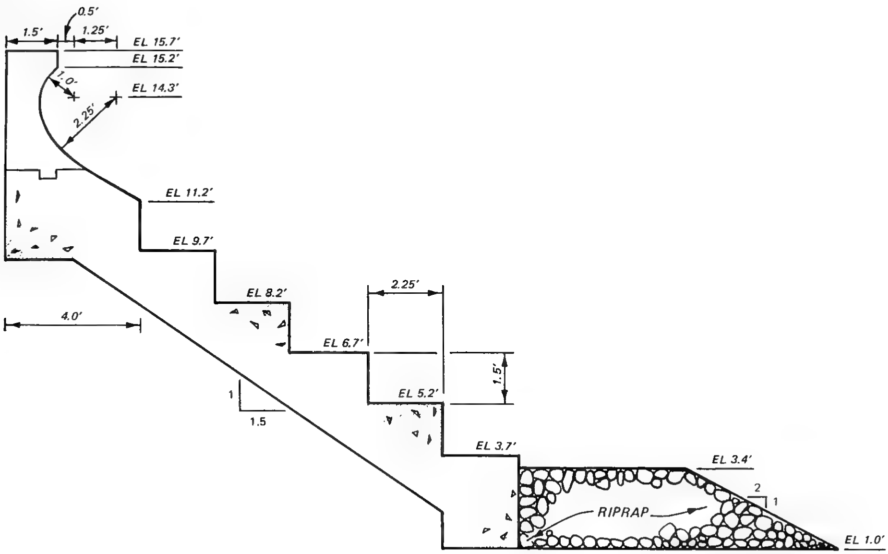
Figure 4. Tank bathymetry and gage arrangement

9. Wave data were collected on eight electrical resistance wave gages. Wave pressures were measured with miniature semiconductor pressure transducers, each equipped with a silicon diaphragm and a four-arm strain gage bridge. Simultaneous pressure measurements were made at six different locations along the face of the seawall (Figure 5). Wave signal generation and data acquisition were controlled using a DEC MicroVAX I computer. Wave and pressure data analysis were accomplished using primarily a DEC VAX 11/750.

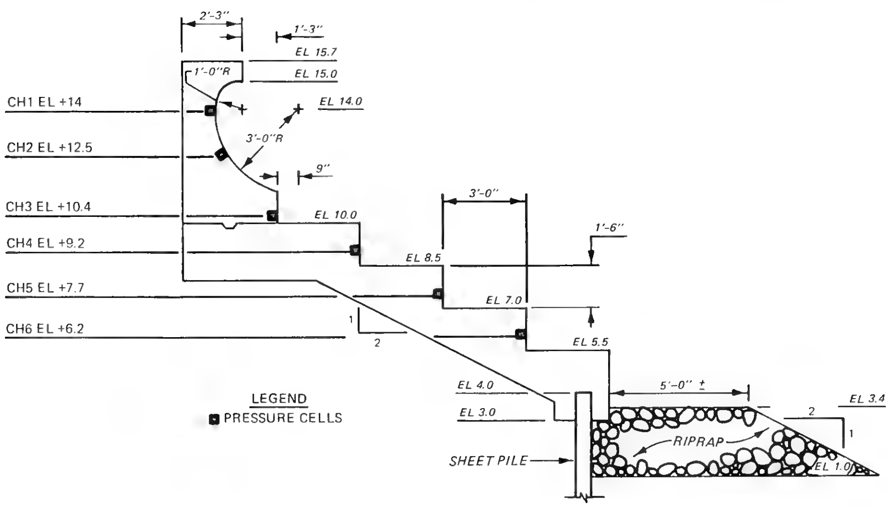
Test Conditions

10. Test conditions were determined based on historical storm records for the Virginia Beach area from 1928 to present. Selection of these conditions involved numerical modeling of three hurricanes and three northeasters which were chosen as the most severe storms in the historical record (Lillycrop, Pope, and Abel, in preparation). Portions of the wave hindcast data came from the Sea State Engineering and Analysis System (SEAS). The remainder were obtained from existing Wave Information Studies (WIS). After a data base was established, all wind wave computations were made using the WIS discrete spectral wave transformation model. This procedure used three hurricane storms and three northeaster storms which were considered representative of the worst storms on record. From these six storms, the most significant of each type was chosen to be represented in the physical model. The test storms were, specifically, the hurricane of August 1933 and the northeaster of March 1962. These are the most severe storms of record for Virginia Beach, and they were generated using TMA spectra, which are analytical spectra representing the depth and frequency transformations of a deep-water wave moving into shallow water (Hughes 1984 and Lillycrop, Pope, and Abel, in preparation). A description of the design deepwater wave conditions reproduced in the tank is provided in Table 1 for the zero-moment wave height H_{mo}, wave period T_p, and three spectral shape parameters γ, σ_{lo}, and σ_{hi}. Swl's were chosen to bracket historical storm surge elevations, and those selected for testing were +6.0 ft, +7.0 ft, +8.0 ft (project design water level), and +9.5 NGVD.

11. The wave machine was calibrated by generating monochromatic waves of differing heights and periods while measuring these waves at various gage locations in the tank. In the same manner, spot checks of the TMA spectra



a. Phase I



b. Phase II

Figure 5. Profile view of seawall

Table 1

Design Wave Conditions

Storm Type	TMA Spectral Parameters				
	H_{mo}, ft	T_p, sec	γ	σ_{lo}	σ_{hi}
Hurricane	15.81	13.7	1.1	0.0001	0.90
Northeaster	13.60	15.4	3.4	0.1300	0.15

were made to verify that the required spectra were being reproduced in the wave tank at the wave board.

Model Construction

12. Model seawalls for overtopping tests were constructed by covering a 1.5-ft-wide marine plywood frame with sheet metal. For the pressure tests, an additional 8-in.-wide center section was machined from aluminum block to accommodate the six pressure transducers. Profile views of the Phase I and Phase II seawalls are shown in Figures 5a and 5b, respectively. Locations of the pressure transducers are also shown in Figure 5b.

13. Figure 4 shows a plan and profile view of the tank bathymetry, gage locations, overtopping basins, etc. Overtopping rates were determined by measuring the change in water levels in two containers located behind the seawall.

14. During testing of the Phase I seawall, stability of a proposed fronting stone riprap revetment was investigated. Sizing of the model stone was accomplished using the following transference equation:

$$\frac{\left(W_a\right)_m}{\left(W_a\right)_p} = \frac{\left(\gamma_a\right)_m}{\left(\gamma_a\right)_p}\left(\frac{L_m}{L_p}\right)^3\left[\frac{\left(S_a\right)_p - 1}{\left(S_a\right)_m - 1}\right]^3$$

where

W_a = weight of an individual stone, lb

m,p = model and prototype quantities, respectively

γ_a = specific weight of an individual stone, pcf

$\frac{L_m}{L_p}$ = linear scale of the model

S_a = specific gravity of an individual stone relative to the water in which it was placed, i.e. $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of water, pcf

PART III: WAVE OVERTOPPING INVESTIGATION

Testing Procedures

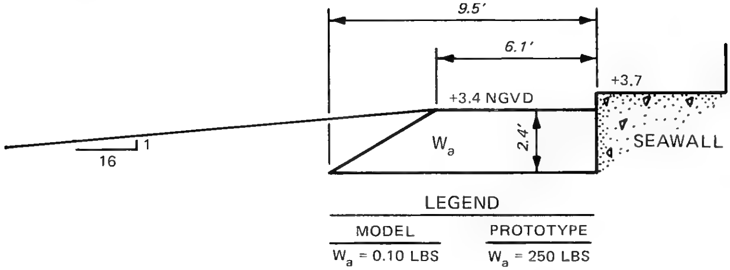
15. A typical test run for collecting wave overtopping rates took place as follows. Wave gages were calibrated at the beginning of each day of testing. The proper signal generation file was loaded into the data acquisition program, and a percent gain was selected. (Percent gain varies the wave height H_{mo} at the wave board without changing the peak period T_p or phasing.) Initial water level readings in the two overtopping containers were recorded, and generation of the wave field was begun. During the following 30 min of testing, water from the lower overtopping container was pumped into the upper container, quantified, and released back into the flume as necessary. This procedure minimized the effect that removal of overtopped water might have on swl and wave conditions. When a test was completed, final water level readings were taken, and the water surface in the flume was allowed to still before another test run was started.

16. The wave gages acquired data at 20 samples per second and, for the majority of test runs, wave data and overtopping measurement were collected throughout the entire 30-min run. For the range of conditions tested, the zero-moment wave height H_{mo} near the structure varied from about 3.5 to 6.0 ft, and the peak period T_p near the structure varied from about 10.0 to 20.0 sec (see Appendixes A and B).

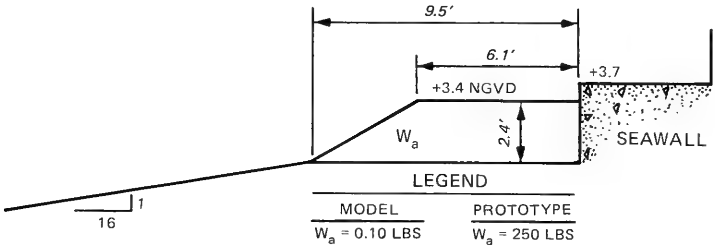
17. Stability of the toe armor stone was observed during each overtopping test condition at each of the swl's. Results of the stability tests and overtopping quantities were recorded by an experienced technician, and selected events were documented by still photography and video footage.

Riprap Stability

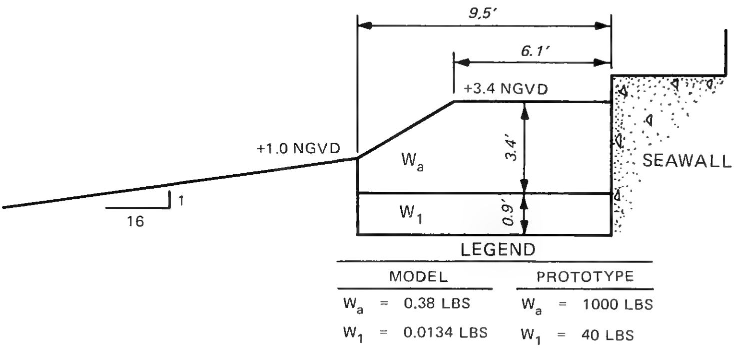
18. In an attempt to control overtopping by restricting the scour depth that influences overtopping, a riprap fronting berm was proposed for the Phase I seawall design at the initiation of the model. The initial proposal by CENAO dictated determination of overtopping rates and stability of riprap toe at the +3.4 NGVD elevation which would have left the riprap unexposed to wave attack (Figure 6a). To adequately determine berm stability, it was



a. Unexposed riprap toe as originally proposed



b. Exposed riprap toe as tested



c. Stable riprap toe as determined by model tests

Figure 6. Riprap toe as proposed and tested

recommended that the toe stone be exposed to wave attack; therefore, the wall was tested with a fronting slope intersecting the toe of the riprap at +1.0 NGVD (Figure 6b).

19. Initially, a berm with a median stone weight of 250 lb (Figure 5b) was tested and found to be unstable (Photo 1). Based on Goda's stability theory (Goda 1985 and Tanimoto 1982) and engineering judgment, a 1,000-lb median stone weight (Figure 6c) was selected for testing. Stability of this berm was acceptable for all test conditions (Photo 2).

20. Observations made during the tests suggested that the fronting riprap toe reduced wave overtopping, especially at swl's of +7.0 ft and less. This conclusion is based on the observed energy dissipation as waves propagated over the berm at the lower swl's. A quantitative description of the influence of the fronting riprap on overtopping rates at low water levels is difficult; however, it is apparent that at swl's of +8.0 ft and above the fronting riprap caused little reduction of overtopping rates at the seawall. To accomplish a significant decrease in the overtopping rates, the berm width would have to be increased considerably before its dissipating effect on the long-period storm waves which were tested would be noticeable. Nevertheless, presence of the 10-ft-wide fronting berm could be advantageous in other respects. The riprap may help minimize undermining at the toe of the structure and could help to reduce erosion of the beach adjacent to the structure by absorption of incident wave energy. Since the Virginia Beach seawall was designed with a steel sheet-pile cutoff wall to prevent undermining of the structure, inclusion of a fronting riprap berm may not be necessary.

Analysis of Overtopping Parameters and Trends

21. The dimensionless relative freeboard parameter which consolidates the data into a single trend was first developed and used for the Roughans Point seawall/revetment study for US Army Engineer Division, New England (Ahrens and Heimbaugh 1986). The relative freeboard parameter is defined as follows:

$$F' = \frac{F}{\left(H_{mo}^2 \; L_p\right)^{1/3}} \qquad (1)$$

where

F = average freeboard, or that distance between the crest of the seawall and the local mean water level

H_{mo} = zero-moment wave height measured at Goda Array 2 (wave gages 5, 6, and 7) and assumed to be representative of the H_{mo} at the toe of the structure

L_p = significant wave length associated with peak period T_p measured at Goda Array 2 and computed using Hunt's method (Hunt 1979)

The relative freeboard parameter F' can be thought of as the ratio of freeboard to severity of local wave climate. As wave climate becomes more severe, F' becomes smaller until a point is reached when the wall is being inundated with waves such that the energy dissipation through wave/structure interaction is insignificant. To establish data trends for the Phase I and Phase II seawalls, the relative freeboard parameter was plotted versus the measured overtopping rate Q in cubic feet per second per foot (cfs/ft) of seawall.

22. The Phase I and Phase II tests were limited to a relatively narrow band of wave conditions because only two peak periods, and corresponding maximum wave heights, were specified in the selected design events. To better establish data trends and cover a wider range of possible storm conditions, wave heights of the wave board were varied for the two specific wave periods at each of the selected swl's (see Appendix A). Figures 7 and 8 show that as the percent gain of the design wave height at the wave board was increased, the wave energy of the spectrum, or H_{mo}, measured at Goda Array 2, approached an approximate limiting value (Hughes 1984). This theoretical approximate limiting value of H_{mo} is controlled by the water depth and is calculated by

$$H_{mo_{(max)}} = B\ h \tag{2}$$

where B is dependent on the fronting beach slope and typically ranges from 0.55 to 0.65, and h is the water depth. A value of $B = 0.6$ is suggested for a typical beach slope and was used along with the water depth and associated setup to plot the limiting value lines seen in Figures 7 and 8. These plots indicate that the maximum amount of energy for a particular water depth was reached at the higher percent gains.

23. The Phase I seawall (Figure 5a) was initially tested for hurricane and northeaster storm events, as previously described, for swl's of +6.0 ft,

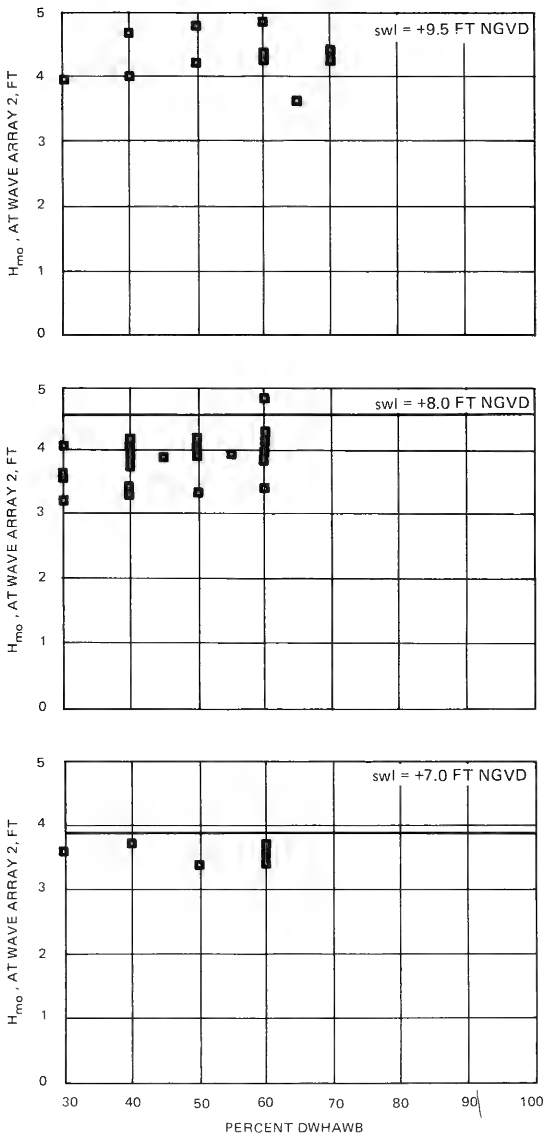


Figure 7. H_{mo} vs percent gain for Phase I seawall at swl's of +9.5, 8.0, and 7.0 ft NGVD (DWHAWB = design wave height at the wave board)

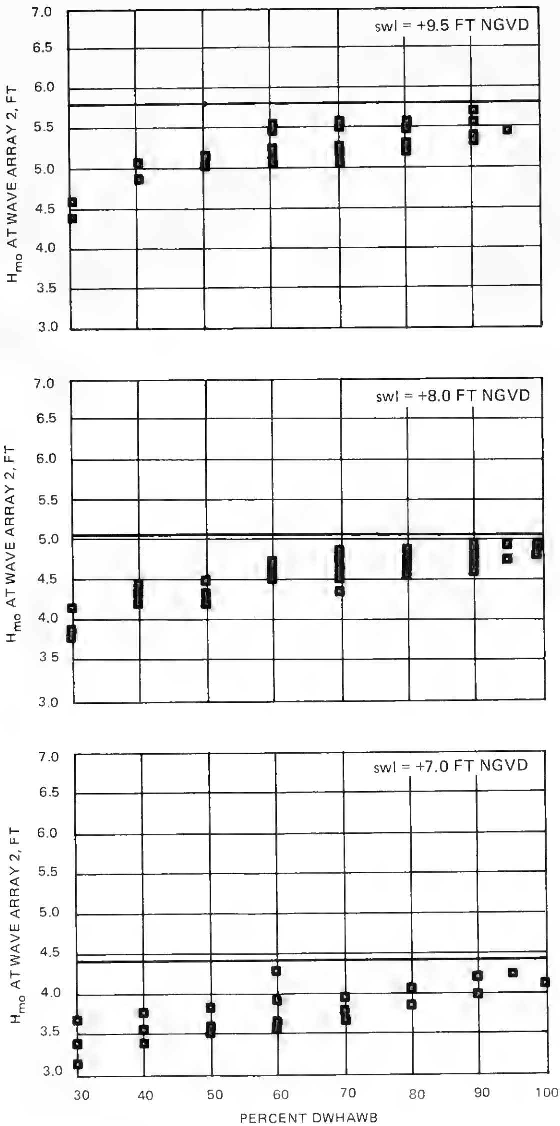


Figure 8. H_{mo} vs percent gain for Phase II seawall at swl's of +9.5, +9.0, and +7.0 NGVD

+8.0 ft, and +9.5 ft. Since no significant overtopping occurred at +6.0 ft swl, the minimum water level was raised to +7.0 ft NGVD. A data plot of Q versus F' showing results of the Phase I seawall tests is presented in Figure 9. Detailed test data are tabulated in Appendix A, Table A1. The data presented in Figure 9 show a definite trend which can be defined by the following general equation:

$$Q = Q_o \exp (C_1 F') \tag{3}$$

where

Q = overtopping rate, cfs/ft

Q_o = regression coefficient, cfs/ft

C_1 = dimensionless regression coefficient

F' = dimensionless relative freeboard parameter

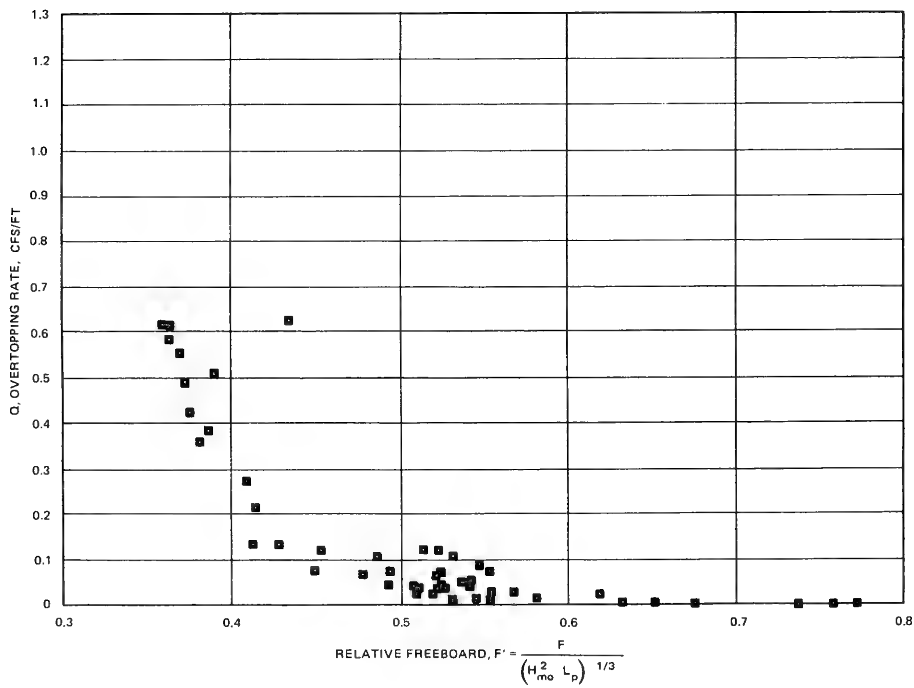


Figure 9. Q versus F' data plot for Phase I seawall

This general equation not only includes the incident wave height and period, water depth, and seawall freeboard but also provides a means for comparing seawall performance and predicting percent differences in the overtopping rates for various beach erosion levels in front of the structure. Such comparisons will be made later in this report.

24. CENAO indicated that the overtopping rates measured for the Phase I seawall were not satisfactory and requested suggestions of how the overtopping could be reduced. Suggestions considered were to: (a) increase the crest elevation of the wall, (b) place a large revetment in front of the wall, and (c) change the geometry of the seawall). Item (a) was believed to be the most promising but was rejected by CENAO because of local community objections. Placement of a large revetment in front of the wall was deemed impractical and uneconomical; thus, it was recommended that the geometry be changed by adding a lip, or extension, to the recurved portion of the seawall. This alternative was agreeable with CENAO, and a modified seawall geometry developed by CENAO, Phase II seawall, was constructed for testing.

25. The Phase II seawall (Figure 5b) was tested using hurricane and northeaster storm events for swl's of +7.0, +8.0, and +9.5 ft NGVD. All data generated from the Phase II seawall tests (including data from wave heights of 30 to 100 percent of DWHAWB are presented in Figure 10 and tabulated in Table A2 (Appendix A). Similar to the Phase I seawall test results, these data fit the general trend of Equation 3. Since there was some question whether the prototype overtopping rates for the design events should be based on all the data generated in the Phase II tests (wave heights of 30 to 100 percent DWHAWB) or with only the 100 percent DWHAWB data, a Q versus F' plot of only the 100 percent DWHAWB data for Phase II seawall is presented in Figure 11. This plot contains fewer data points because of the limited number of design events, but the data trend characteristic of Equation 3 is assumed.

Seawall Comparisons

26. To compare the performance of the Phase I and Phase II seawalls, Equation 3 was used. An explanation of Equation 3 and a tabulation of calculated values are presented in Appendix B. Specific comparisons of the percent decrease in Q for hurricane conditions at the three swl's tested are given in Table 2. Since only wave heights up to 70 percent of the DWHAWB at

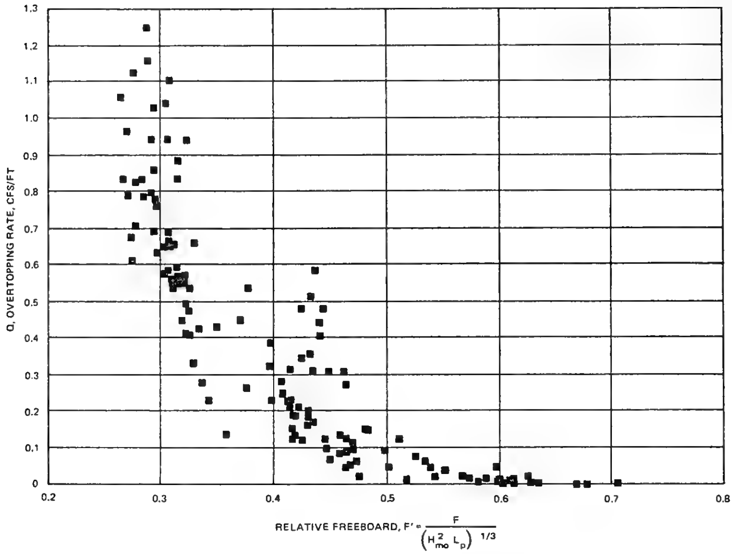


Figure 10. Q versus F' data plot for Phase II seawall

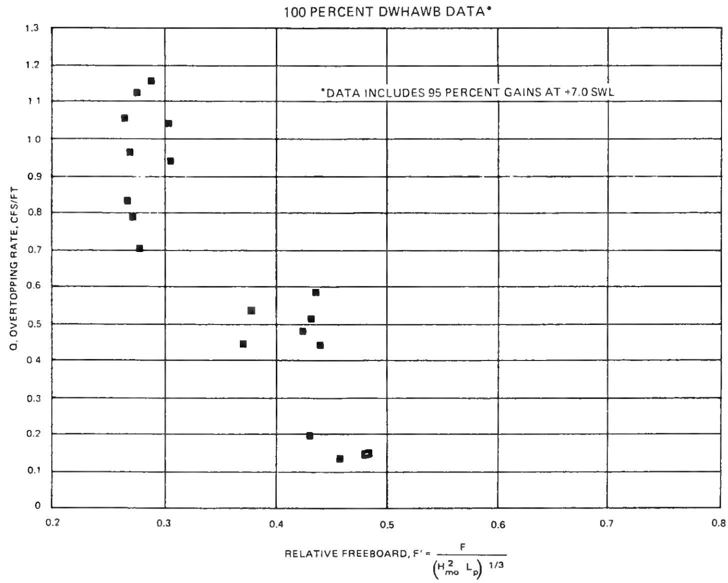


Figure 11. Q versus F' plot using only maximum data point (100 percent gains) for Phase II seawall

Table 2

Phase I Seawall Compared to Phase II Seawall for Hurricane Conditions

swl ft	Percent Decrease in Q Phase I vs Phase II 30 to 70%	Percent Decrease in Phase I vs Phase II 100%
+9.5	43	18
+8.0	48	24
+7.0	54	31

the +9.5 swl and up to 60 percent for all lower swl's could be reproduced in the Phase I seawall tests, relative performance of the Phase II seawall (where wave heights up to 100 percent DWHAWB were reproduced) presented in Table 2 is given first based on 30 to 70 percent data and finally on all data. Based on these comparisons, it can be seen that the seawall geometry changes made from Phase I seawall to Phase II reduced the overtopping rate between 18 and 54 percent depending on the conditions compared. Table 2 also shows that as the swl increased geometry modifications had a smaller effect on reducing the overtopping rate. This occurrence was expected since, as the water level was increased, the waves became larger and began to inundate the wall more often. In short, changes in seawall geometry are less effective at higher swl's.

27. As mentioned earlier, the decision to lower the beach elevation in model tests from +3.4 to +1.0 ft NGVD to test stability of the fronting riprap also affected overtopping rates. For instance, a small change in the depth at the structure toe d_s can significantly affect the magnitude of Q. By lowering the beach elevation, d_s is increased, and this increase in turn affects the local wave length L_p used in Equation 1. Also, as the water depth near the structure becomes deeper, a larger wave can be supported. Thus, as d_s increases, H_{mo} and L_p increase, causing the relative freeboard parameter F' to decrease. As can be seen in Figures 9 and 10, as F' decreases, the overtopping rate Q increases exponentially. Therefore, by decreasing the beach elevation from +3.4 to +1.0 NGVD the overtopping rate is, in effect, increased. Overtopping rates for the +3.4-ft NGVD beach elevation were estimated as explained in the following paragraphs.

28. The relative freeboard versus Q plot was used to predict overtopping rates for extrapolated values of d_s and/or F provided the

projected d_s and F were similar to those tested. There was, however, some question whether the effects of beach erosion should be calculated based on a Q versus F' plot with all the measured data (30 to 100 percent DWHAWB) or based only on the 100 percent DWHAWB measured data. This question was resolved by choosing the more conservative (100 percent DWHAWB) data plot (Figure 11) to calculate the effects of beach erosion for the design storm event where erosion potential is greatest. The method for calculating changes in overtopping rates for the maximum H_{mo} that can exist at each swl is explained, and the respective beach erosion elevations are tabulated in Appendix B. The percent difference and the percent decrease in overtopping rates between the data trend for the Phase II seawall at the +1.0 NGVD beach erosion elevation and the projected trend for the same wall at a +3.4 NGVD beach erosion elevation for hurricane conditions at the three swl's tested are given in Table 3. These numbers are based on Figure 11 where only those data collected at 100 percent DWHAWB were used. The percent difference values in Table 3 were used to predict the change in overtopping for the surge hydrographs (Lillycrop, Pope, and Abel, in preparation).

Table 3

Overtopping Comparisons Using Hurricane Conditions at the +1.0- and +3.4-ft NGVD Beach Elevations

swl, ft	Percent Difference*	Percent Decrease**
+9.5	46	54
+8.0	22	78
+7.0	9	91

* Percent difference in Q for data at +1-ft NGVD beach elevation and projected values at +3.4-ft NGVD beach elevation.

** Percent decrease in Q for beach elevation at +3.4-ft NGVD versus +1.0-ft NGVD. Percents are based on Q values calculated in Table B3 (Appendix B) using only the 100 percent DWHAWB data points.

Wave Setup and Seiche in the Wave Tank

29. In almost all wave tank tests, there can exist local wave setup and seiche. Both of these phenomena can occur in the prototype, but their amplitude and overall effects may not be the same as in model tests; thus, it is important that they be identified and, to the best extent possible, accounted

for in the model. Wave setup is the superelevation of the water surface over normal swl elevations and is due to wave breaking which causes radiation stresses to develop. Seiche is a long-period oscillation which can occur in an enclosed body of water and, in the case of wave tanks, depends mostly on tank length and geometry.

30. It was determined that both wave setup and seiche existed in varying degrees during the Virginia Beach tests. Measured values of wave setup and calculated values of the seiche are reported in the tabulated data (Appendix A). A detailed discussion of wave setup and seiche effects is given in Appendix C. The effects of wave setup were directly accounted for in the model and thus were considered in any subsequent prediction calculations. The main effect of the seiche was that it increased scatter in the data. This was not thought to influence the overall data trend; thus its effect was not included in the data analysis.

PART IV: WAVE-INDUCED PRESSURES INVESTIGATION

31. After completion of the overtopping study, wave pressure tests were performed using the Phase II seawall geometry (model scale 1:19). The purpose of these tests was to obtain pressure data necessary to determine wave-induced forces and moments to which the wall would be subjected under certain storm conditions. Ultimately, this information will be used in the completion of a seawall and foundation design which can withstand expected wave forces and ensure stability against overturning and/or sliding.

32. Wave pressures were measured using miniature semiconductor pressure transducers, each equipped with a silicon diaphragm and a 4-arm strain gage bridge. Pressure measurements were calibrated and recorded using a DEC Micro-VAX I computer. Typical time-histories of measured wave pressures (Figure 12) indicate that as a wave approaches and strikes the face of the seawall, in many cases, it causes an initial shock pressure of large magnitude and short duration immediately followed by a secondary (or surge) pressure of lesser magnitude and longer duration. Based on experiments conducted with a vertical

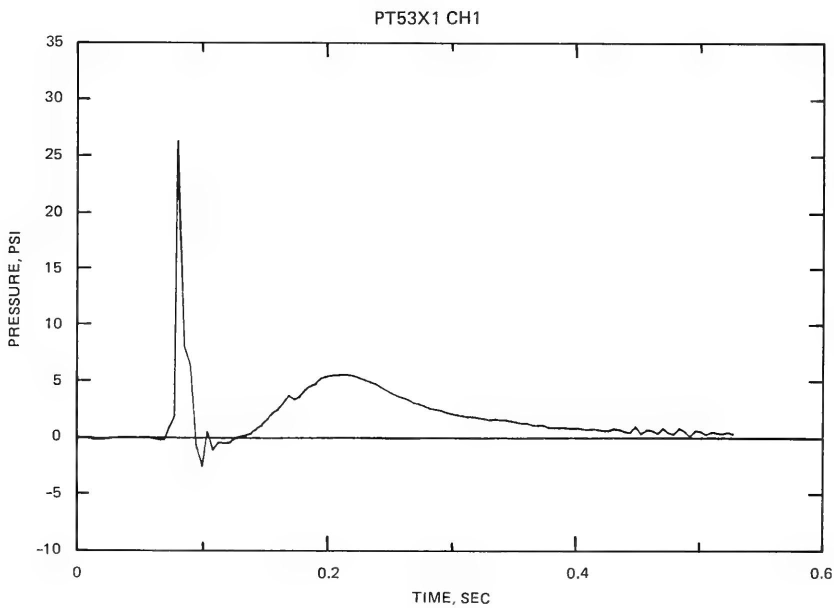


Figure 12. Typical wave pressure time-history

wall, Bagnold (1939) theorized that the short duration shock pressures result from the rapid compression of an air pocket trapped between the face of a breaking wave and the wall. In the past, this phenomenon has been studied by several investigators (Minikin 1946; Carr 1954; Kamel 1968a, 1968b; Garcia 1968; Kirkgoz 1982). However, there is still debate concerning the relative importance of these shock pressures to the actual design of a seawall. A common opinion among many designers is that pressures of such short duration should not be used for establishing design loadings; thus, it is their opinion that the lesser surge pressures of longer duration are more suitable indicators of critical dynamic loadings.

Testing Procedure

33. For the purpose of this study, shock and surge pressures were measured in response to waves characteristic of the same two storms used in the overtopping study. These storms were simulated at swl's of +7.0, +8.0, and +9.5 ft NGVD. Signal generations and resulting zero-moment wave heights were accomplished with gains set at 50 and 100 percent. Test conditions to which the wall was subjected are summarized in Table D1 (Appendix D). Because of limited data storage capacity of the computer facilities used for data acquisition, the duration of each test was dictated by the particular sampling rate at which pressures were measured. As stated above, durations of shock pressures are characteristically quite short (in the range of prototype milliseconds); therefore, to acquire a definitive record of these portions of the pressure response, a high sampling rate was imperative. Tests were initiated using a 2,000-Hz sampling rate which, due to data storage capabilities, limited the actual data acquisition interval to approximately 30 sec. Therefore, with a 2,000-Hz sampling rate, pressure data in response to roughly seven to nine waves in sequence could be obtained. Analyses of these first runs indicated that the 2,000-Hz sampling rate resulted in good resolution of most maximum pressures; however, since the duration of individual tests was so limited (30 sec), a series of tests using various slower sampling rates was undertaken. These tests indicated that an acceptable resolution of most shock pressures could be achieved at a 1,000-Hz sampling rate, thereby increasing the allowable length of each test to 60 sec. Table D1 shows that 16 tests were executed with an 80-Hz sampling rate. These tests were conducted to

allow continuous data acquisition for an entire 30-min run, yielding a more comprehensive time-history of the overall pressure response at the expense of clear resolution of shock pressures.

Overall Results

34. The primary objective of this evaluation of wave-induced pressures was to identify the magnitudes and durations of both the shock and surge pressures on a particular wall geometry. CENAO guidance stipulated that the most important product of this effort would be a series of representative pressure profiles describing some of the more severe conditions encountered. The importance of identifying the occurrence of significant negative pressures was also stressed. In conjunction with these objectives, the presentation of results is concentrated primarily on representative design conditions. Subjection of the seawall to spectral wave conditions resulted in the collection of many less severe but more interesting pressure time-histories; however, detailed analysis of these records is not documented herein. Maximum values recorded on each gage for all runs are listed in Table D2 (Appendix D).

Shock Pressures

35. For each combination of storm, swl, and percent gain, an initial 30-min run (simulating a 2.18-hr prototype) was performed during which the wave train was closely observed and times of occurrence were recorded for the more severe waves (in terms of impact on the seawall). These observations allowed scheduling of 1-min sampling intervals to coincide with the most probable times when maximum pressures would occur.

36. Generally speaking, a 1-min test (simulating a 4.36-min prototype) of a severe condition would provide three to four waves which induced distinct shock pressure records. Most magnitudes of these most severe pressures were in the 20- to 60-psi (prototype) range, although on two occasions pressures as high as 117 psi were recorded. (Throughout the remainder of this text, all values of pressure magnitudes and durations presented will correspond to prototype.) Durations of the most severe shock pressures also varied but to a much lesser extent. Pressures of 15 psi and more were normally characterized by durations of less than 0.020 sec. Durations of the highest pressures

(above 60 psi) were less than 0.010 sec in duration.

37. A typical shock pressure time-history is presented in Figure 13. This particular record was collected on Channel 1 (see Figure 5b) during simulation of the hurricane at a +9.5-ft swl. The peak value measured 105 psi with a duration of approximately 0.038 sec above 10 psi.

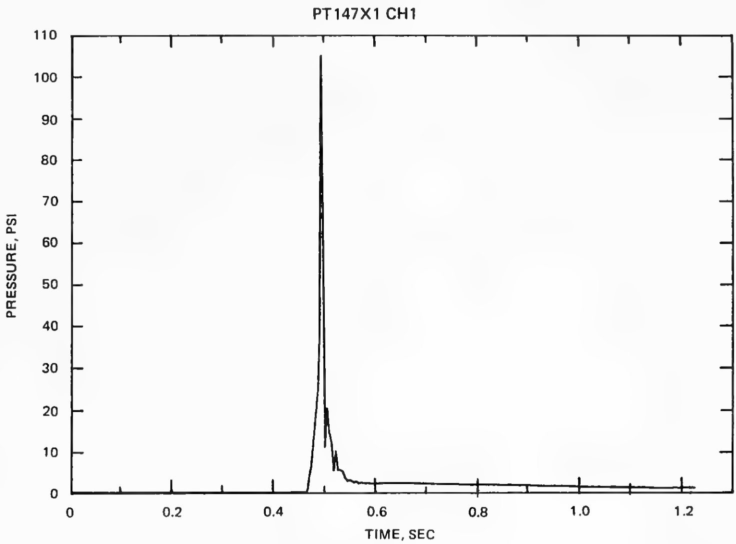


Figure 13. Time-history depicting typical shock pressure

38. At swl's of +8.0 and +9.5 ft NGVD, maximum pressures consistently occurred at Channel 1 which was located near the vertex of the wall curvature. High pressures also were common on the face of the highest step (Channel 3) at these swl's. At the +7.0-ft swl, maximum pressures occurred on the faces of the lower steps (Channels 4, 5, and 6). It is interesting that at no time during data collection did the pressure on Channel 2 exceed 14 psi. Maximum pressures at this location on the wall never displayed characteristics of shock pressures. Instead, they were typified by a well-rounded, relatively small peak of long duration (Figure 14).

39. Plates 1-63 were prepared to provide designers with adequate information concerning pressure profiles in response to severe wave conditions. For each location of a pressure transducer, records containing the five greatest pressures encountered were retrieved and analyzed in greater detail. Six

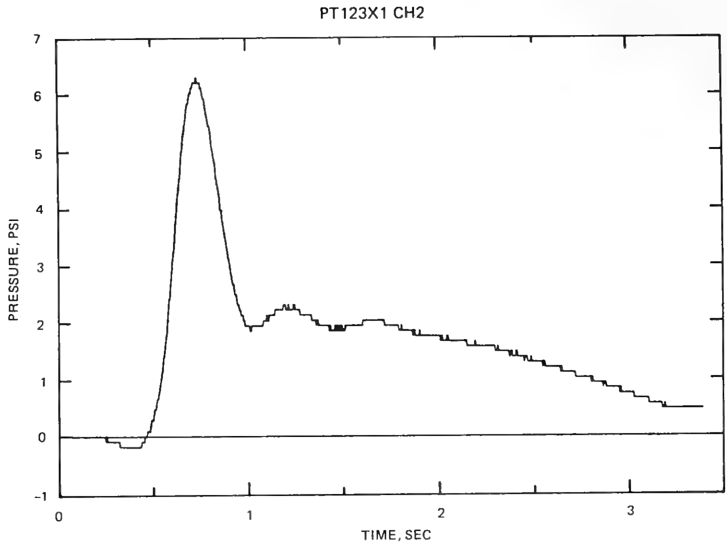


Figure 14. Typical maximum pressure record for Channel 2

profiles exist for each of these records. These six instantaneous profiles represent the six points in that particular record when a maximum value was occurring on one of the transducers. For example, Plates 1, 2, and 3 depict the six points in time when maximums were occurring during Test 45 (NE, swl = +9.5, 50 percent gain). The plot labeled PT45 MAX CH2 is an instantaneous pressure profile at the point in time during Test 45 when the highest pressure on Channel 2 was monitored. All other profile plates are labeled accordingly. The pressure distributions indicate that maximum pressures at different wall elevations rarely occur simultaneously, especially in the case of a nonvertical wall such as the stepped wall studied here, on which some wave energy is dissipated through turbulence. Notably, the profiles often depict surge pressures on channels other than the one experiencing a maximum. For example, profiles labeled PTXXX MAX CH1 represent the instant in time when the wave has reached the last instrumented point on the face of the seawall. Therefore, impact loads on the more seaward transducers occurred earlier, if at all, and longer duration surge pressures are actually being measured at that point. It should also be noted that the negative pressures indicated on Plates 4 and 5 resulted from a mistaken zero offset before the test was performed.

Surge or Secondary Pressures

40. Results of this study indicate that although surge pressure magnitudes were very consistent (at about 5 to 10 psi for the more extreme conditions) the durations could be quite variable. This phenomenon is primarily related to defining surge pressures, and the variation corresponds to expected results since simple observation of the wall when subjected to spectral wave conditions reveals that the mass of water on a particular wall location varies a great deal throughout a series of waves. However, the most typical surge pressure durations were in the 2.0- to 3.0-sec range (Figure 15). These most

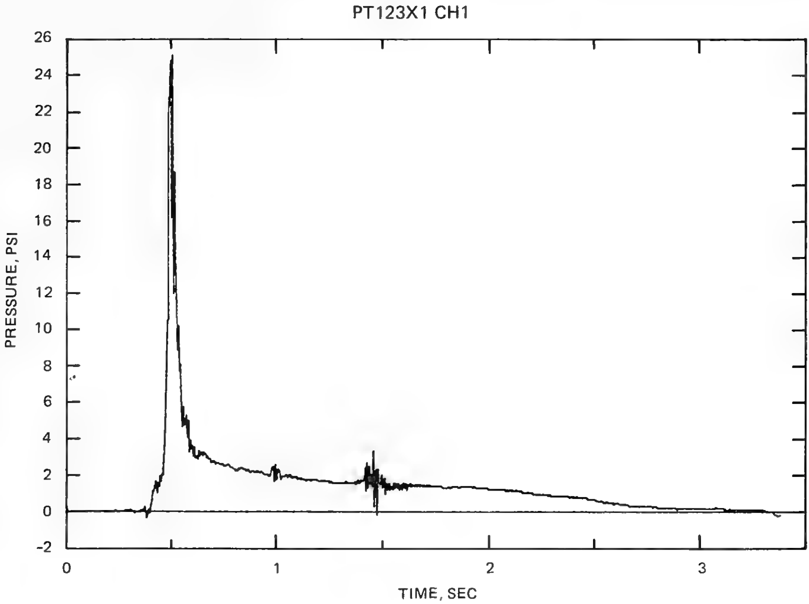


Figure 15. Time-history showing typical surge pressure

distinct surge pressures, in all cases, were recorded immediately after a significant shock pressure. Since little variation actually existed in the profile distributions of the surge pressures, numerous plots of this type were not included. A typical surge pressure profile measured during the northeaster at a +8 ft swl is shown in Figure 16.

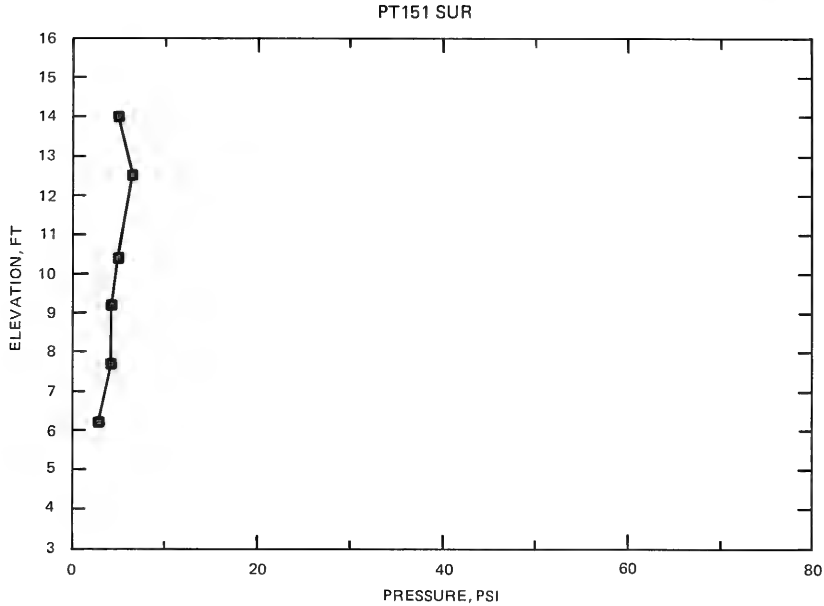


Figure 16. Typical instantaneous profile of surge pressures

Negative Pressures

41. As stated previously, CENAO personnel had expressed an interest in identifying significant negative pressures experienced during testing. Primary interest was related to whether wave runup or drawdown could induce negative pressures small enough to warrant inclusion in the procedure for calculating design uplift forces.

42. A cursory analysis of the data indicated that significant negative pressures may have been recorded. Ten records included measured pressures with values less than -20 psi. However, closer inspection of these records indicated that the small negative pressure durations were less than 1 msec. Also, in most cases the minimum negative pressures occurred within milliseconds of a maximum shock pressure. Such events are shown in Figures 17 and 18. Due to extremely small durations characteristic of these events, these records were not evaluated in further detail; therefore, at this time an explanation of these occurrences is incomplete. In all cases, these events

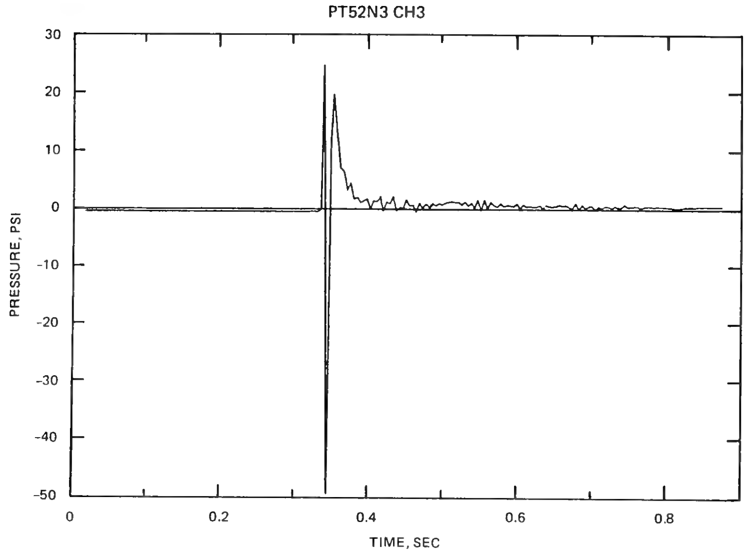


Figure 17. Record depicting minimum negative pressure, Channel 3

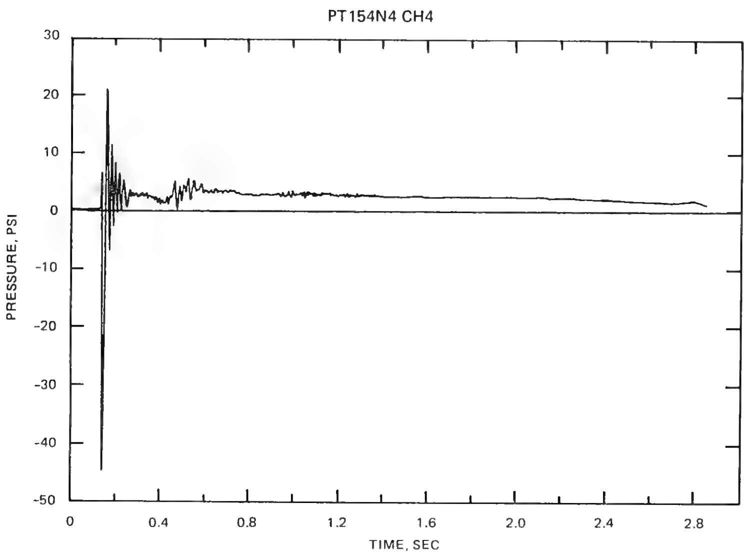


Figure 18. Record depicting minimum negative pressure, Channel 4

occurred at the lowest three transducer locations during events with a high swl. It is suspected that this may be a characteristic of turbulence and air entrainment occurring at the base of each seawall step. Analysis of all other data files failed to identify the occurrence of significant negative pressures.

PART V: CONCLUSIONS

43. Based on the 2-D physical model test results reported herein, it was concluded that:

- a. Regarding wave overtopping and berm stability tests for the storm conditions to which the structures were subjected:
  - (1) The berm design characterized by stone weights of 250 lb was not acceptable in terms of stability of the riprap structure.
  - (2) The berm design using 1,000-lb stones was acceptable.
  - (3) Visual assessment indicated that the riprap toe played an important role in reducing overtopping at swl's of +7.0 ft or less and a lesser role at the +8.0- and +9.5-ft swl's.
  - (4) Overtopping rates measured with the Phase II seawall geometry in place were less than corresponding rates measured with the Phase I design.
  - (5) Overtopping rates observed with a +1.0-ft beach elevation can be expected to decrease by as much as 78 percent for a hurricane event at a +8.0-ft swl with a beach elevation of +3.4 ft.
  - (6) Much of the data scatter in the overtopping results seems to be caused by the occurrence of seiche in the wave flume.
- b. Regarding wave pressure testing:
  - (1) Shock pressures as great as 117 psi were recorded; however, durations of pressures greater than 15 psi were typically less than 0.020 sec.
  - (2) At swl's of +8.0 and +9.5 ft, maximum pressures consistently occurred at the vertex of the wall curvature. Highest pressures were also common on the face of the topmost step at these swl's.
  - (3) At the +7.0-ft swl, maximum pressures occurred on the faces of the lowest three steps.
  - (4) Secondary pressure magnitudes were relatively consistent at approximately 5 to 10 psi. Durations of significant secondary pressures ranged from 2.0 to 3.0 sec.
  - (5) No significant durations of negative pressures were recorded. Design calculations for uplift pressures on the Phase II seawall may be performed neglecting any contribution due to wave runup or recession.

44. Relative to wave overtopping, results of this model study indicate that the Phase II seawall geometry is a more effective design of the two alternatives tested. At the +8.0-ft swl, overtopping rates measured during

Phase II testing were 24 to 48 percent less than corresponding rates measured with the Phase I seawall in place.

45. At the higher water levels of +8.0 and +9.5 ft, the riprap fronting berm appeared visually to have less influence on the reduction of overtopping rates; however, general observations with and without the berm in place indicated that the structure did reduce overtopping at the +7.0-ft swl. Without further tests it is hard to say how much the overtopping rates would be affected, without the berm in place, at the higher water levels. Also, the presence of the berm could help to reduce beach scour at the seawall by helping to dissipate incident/reflected wave energy. Tests indicated that 1,000-lb stone were of adequate size to ensure berm stability under the storm conditions tested.

46. Wave setup and seiches did occur in the wave flume, and these phenomena were considered during data analysis. The seiche influence was responsible for much of the scatter evident in the presentations of the overtopping data. This influence was not great enough to affect overall data trends.

47. Results of the pressure tests indicate that wave pressures in excess of 100 psi can be experienced on the seawall under severe wave conditions; however, these pressures in excess of 15 psi characteristically have durations of less than 20 msec. The question remains--at what duration can a designer confidently establish a threshold above which pressure magnitudes are considered of serious importance? Presently, the answer is a matter of personal opinion. Some individuals (Carr 1954, Cole 1972, Garcia 1968, and Ross 1953) who have investigated this problem feel that the lesser secondary pressures of longer duration are more critical for designer purposes. These particular tests identified secondary pressures with magnitudes of approximately 5 to 10 psi and durations in the range of 2.0 to 3.0 sec.

48. Although the geometry of the seawall prevented installation of a vertical transducer in the extreme upper curvature of the wall, the area did not appear to be subjected to large pressures. Visual assessment of the testing indicated that this was not an area where wave energy was being concentrated. Similar tests performed on the Cape Hatteras Lighthouse seawall indicated that this was an area of concern due to the pressure magnitudes measured on the overhang (Grace and Carver 1985). However, that particular design incorporated a 2-ft extension to the original overhang. In comparison, the lack

of wave energy concentration in this area on the Virginia Beach seawall is due to the milder curvature and relatively small overhang.

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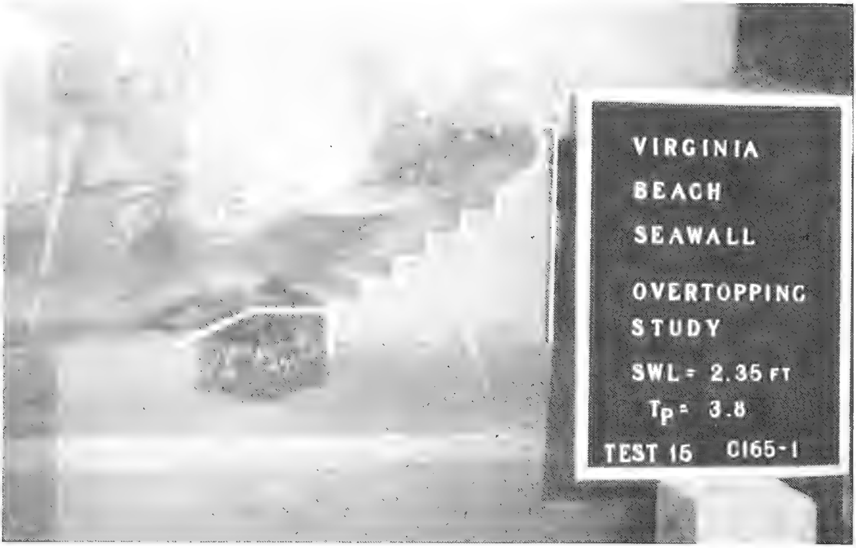


Photo 1. Unstable riprap, 250-lb median weight stone
Phase I, hurricane, swl = +6.0 ft

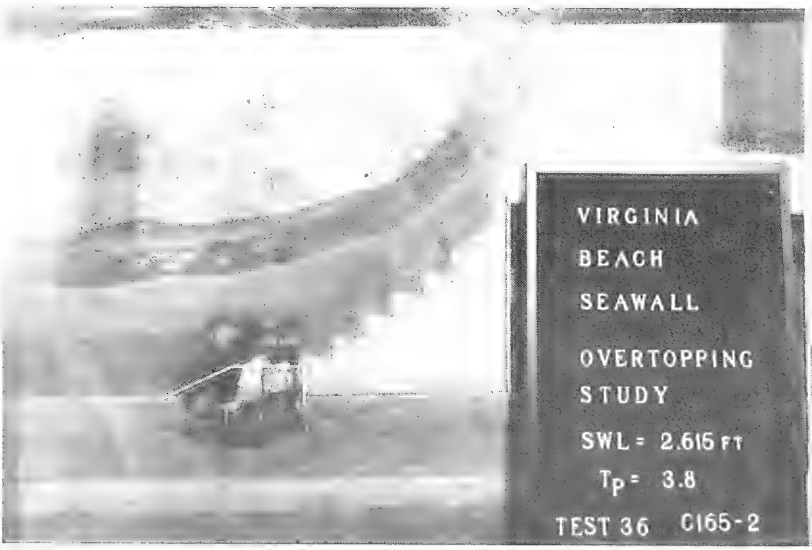
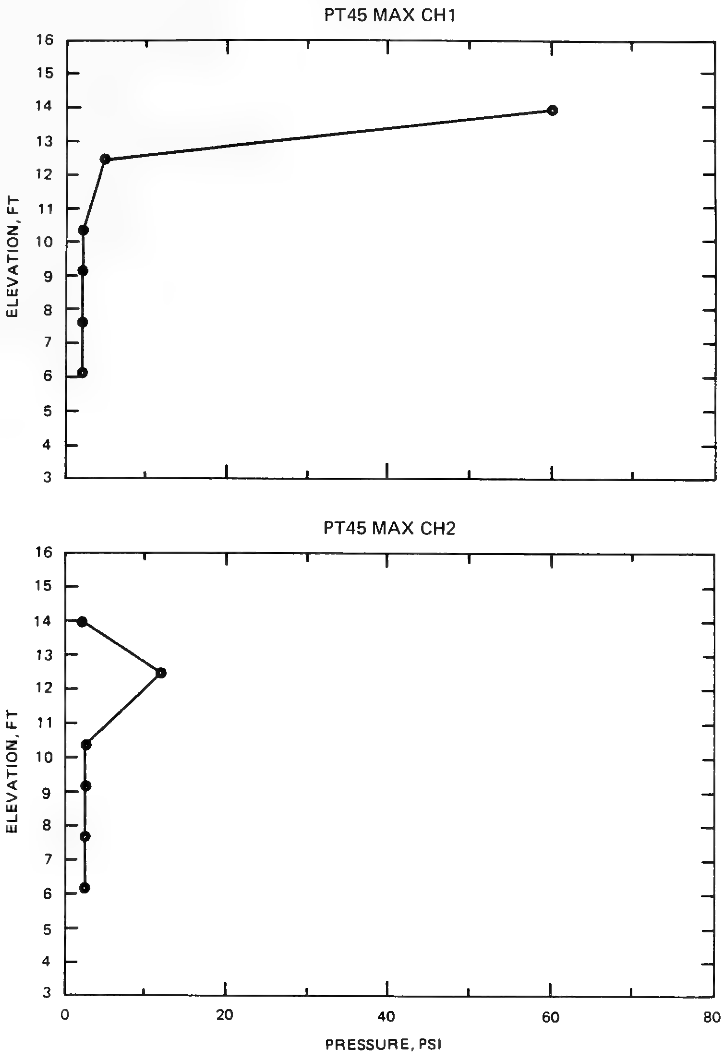


Photo 2. Stable riprap, 1,000-lb median weight stone
Phase I, hurricane, swl = +9.5 ft

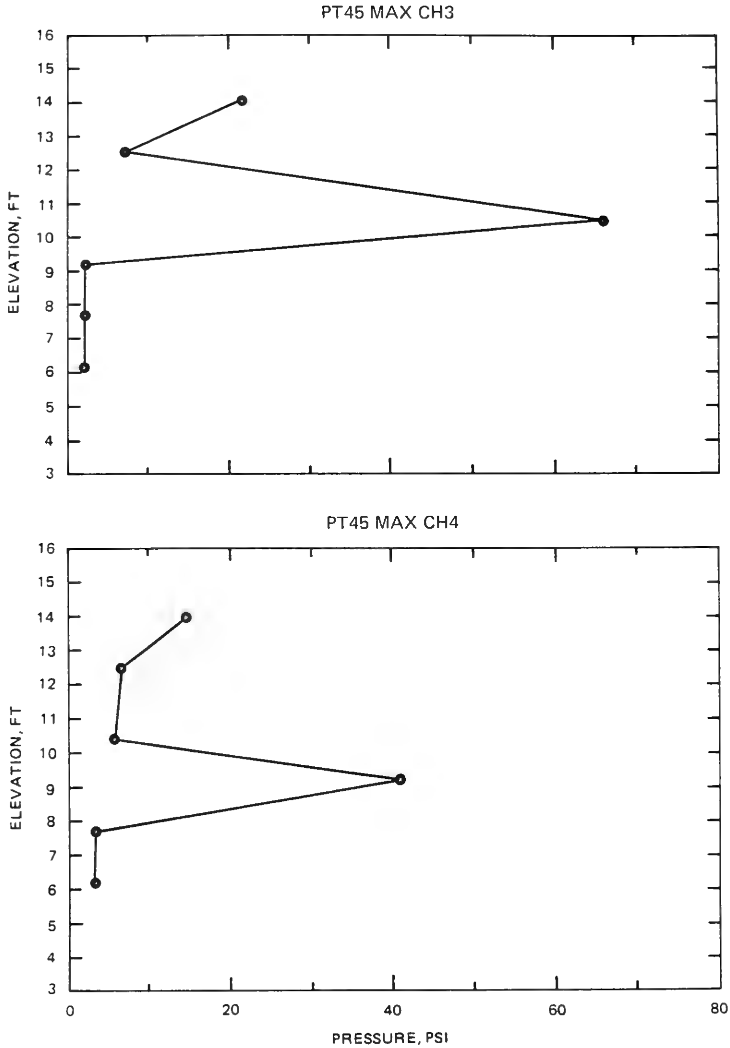


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT45

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

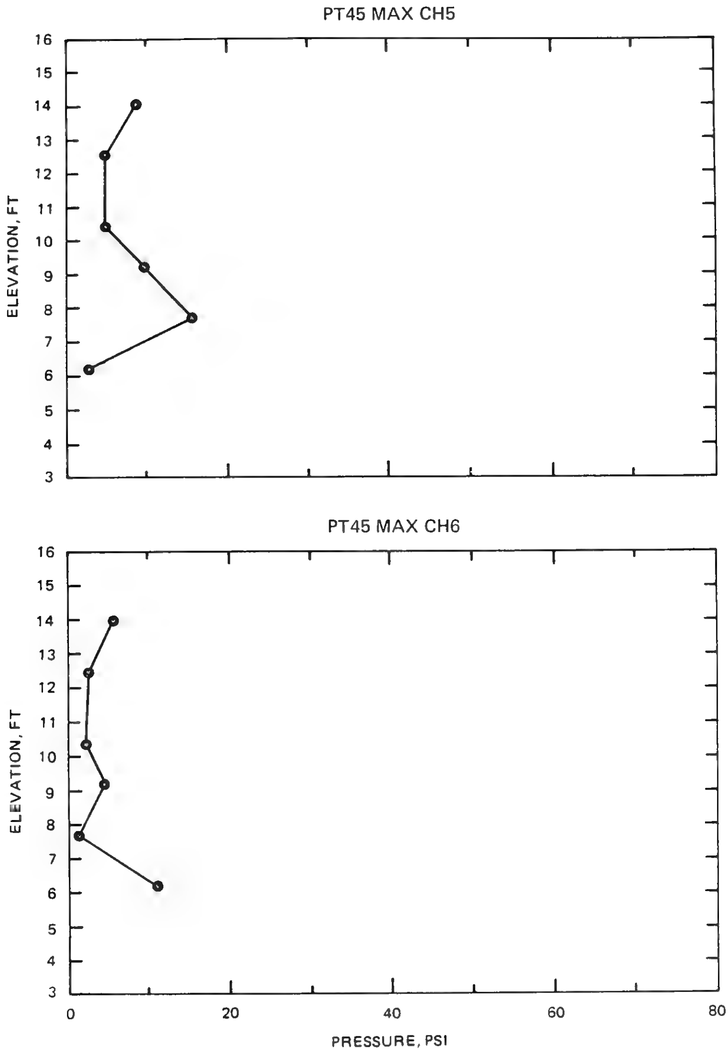


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT45

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

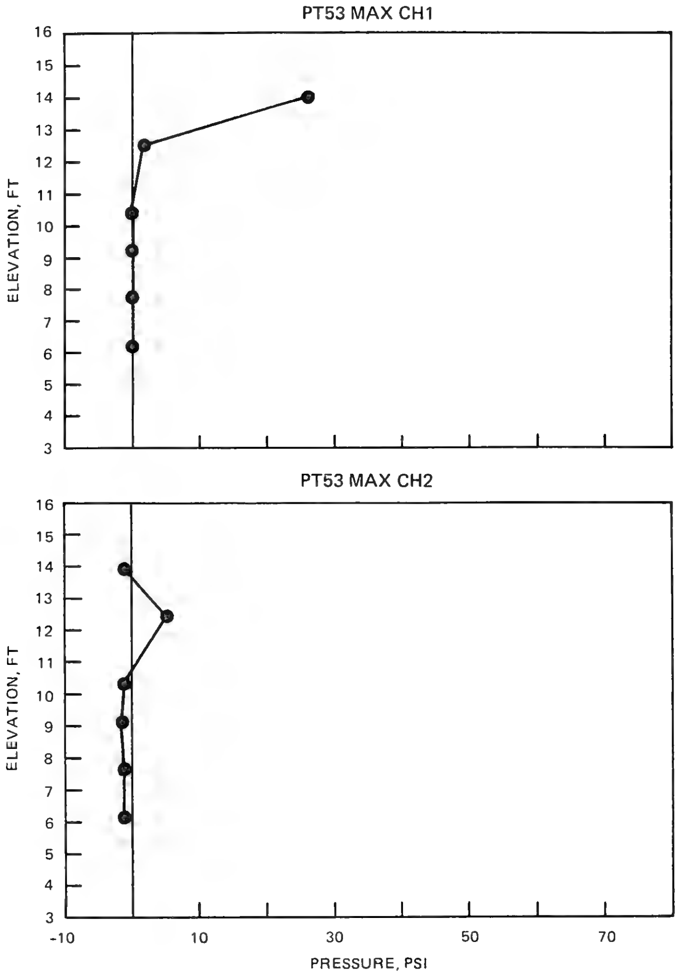


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT45

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

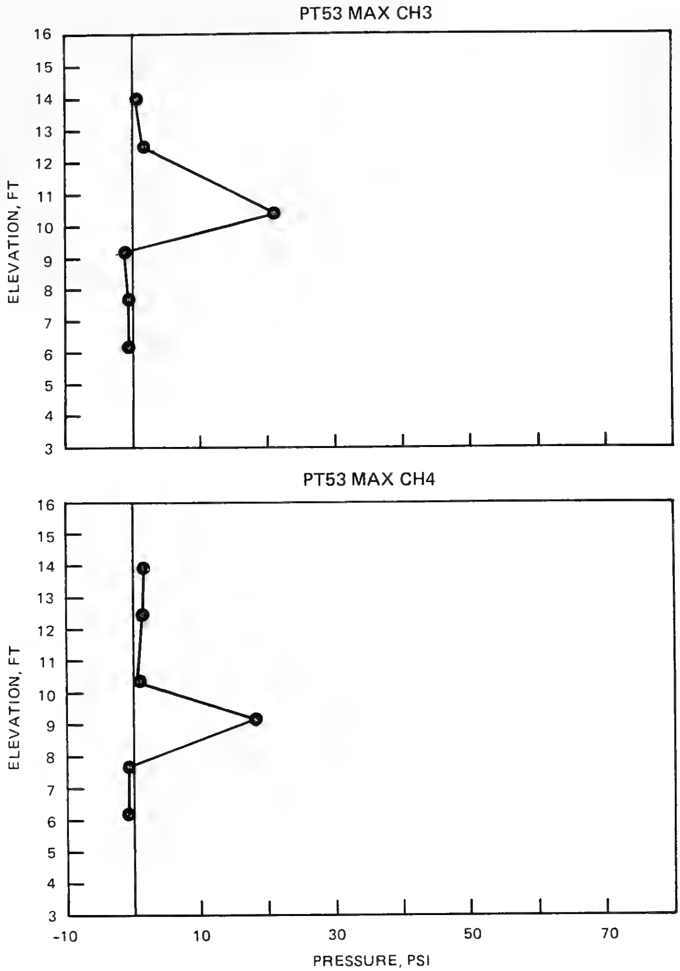


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT53

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

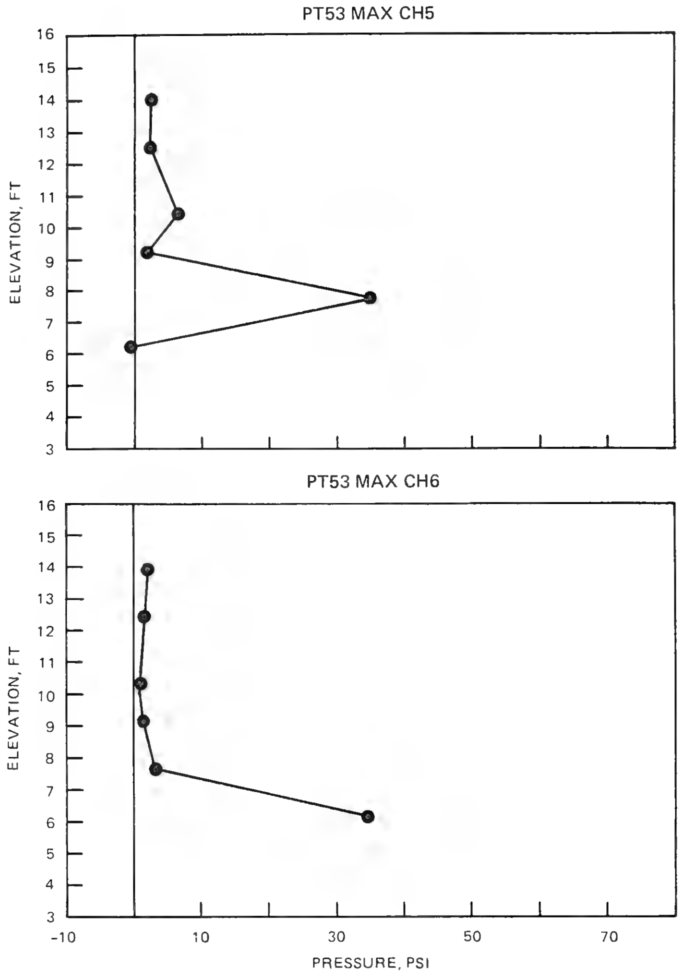


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT53

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

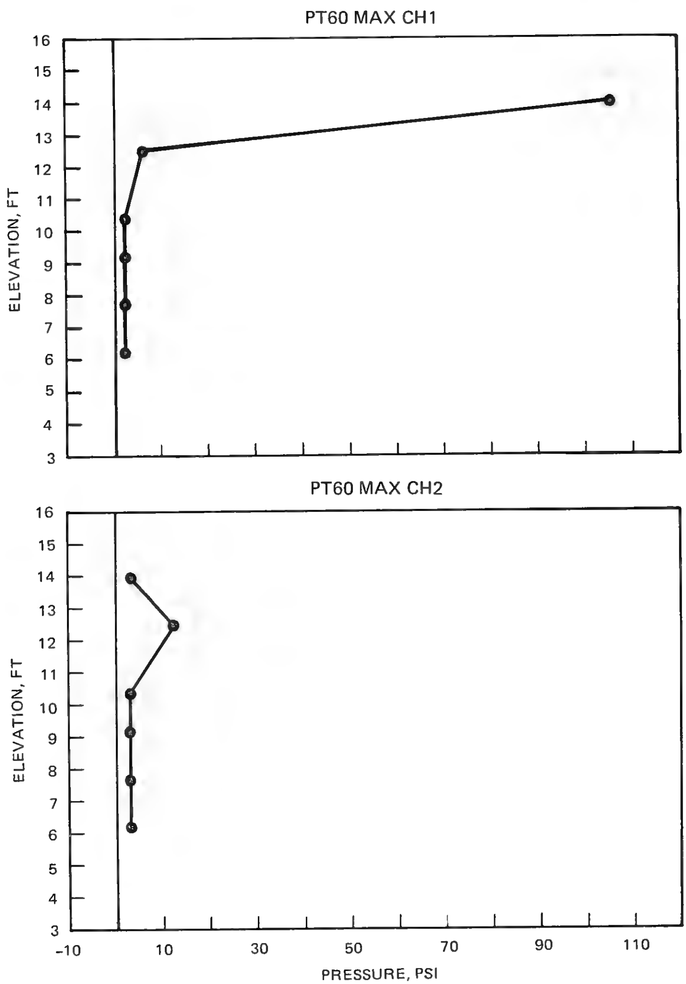


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT53

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6



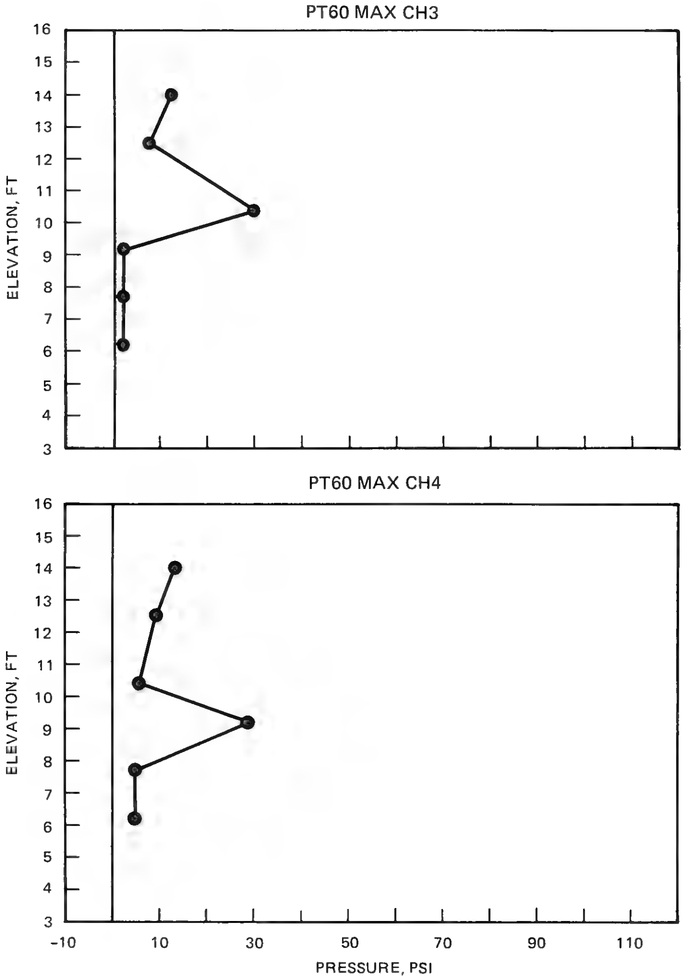
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT60

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

PLATE 7

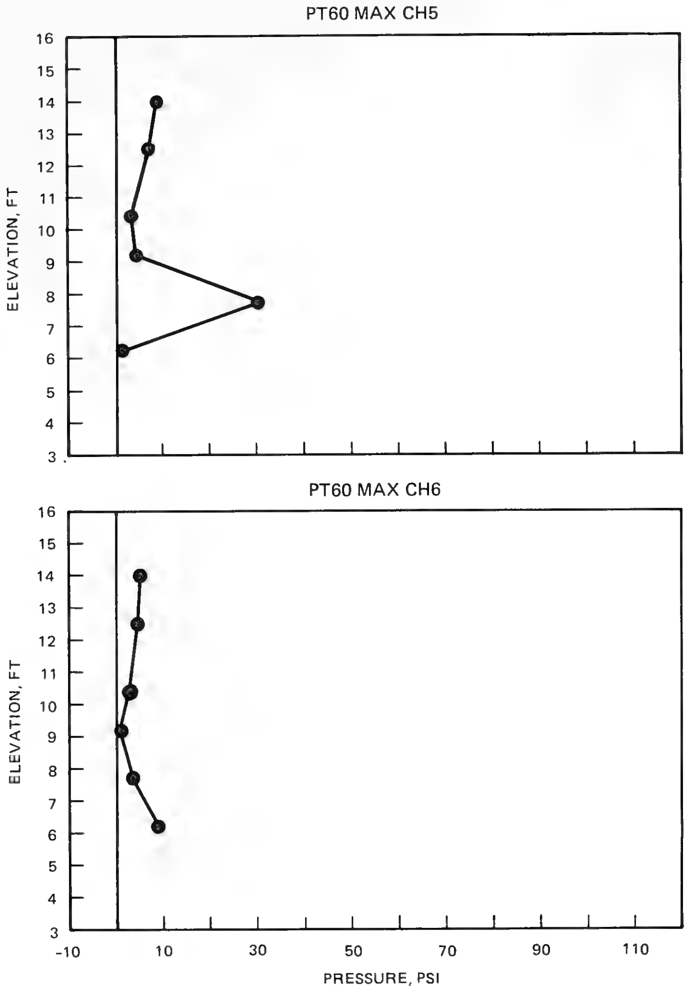


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT60

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

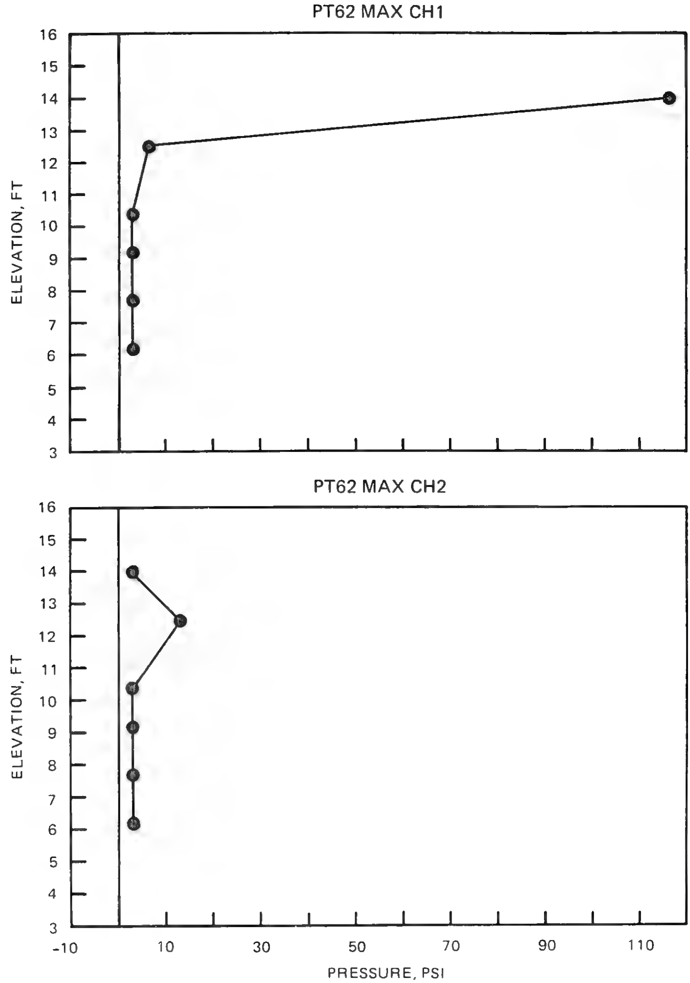


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT60

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

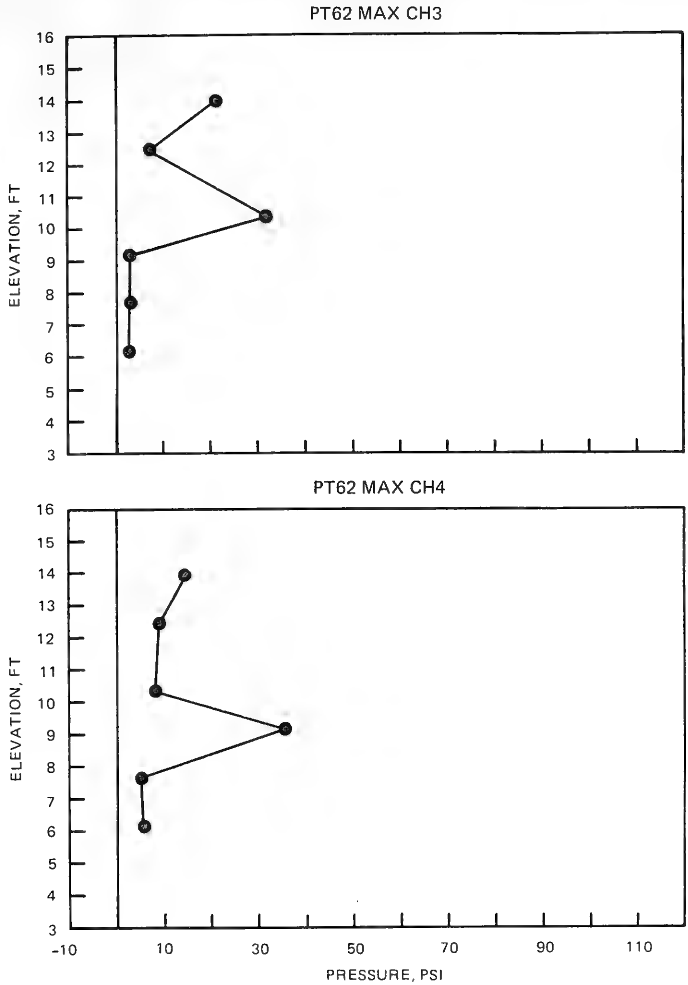


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT62

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

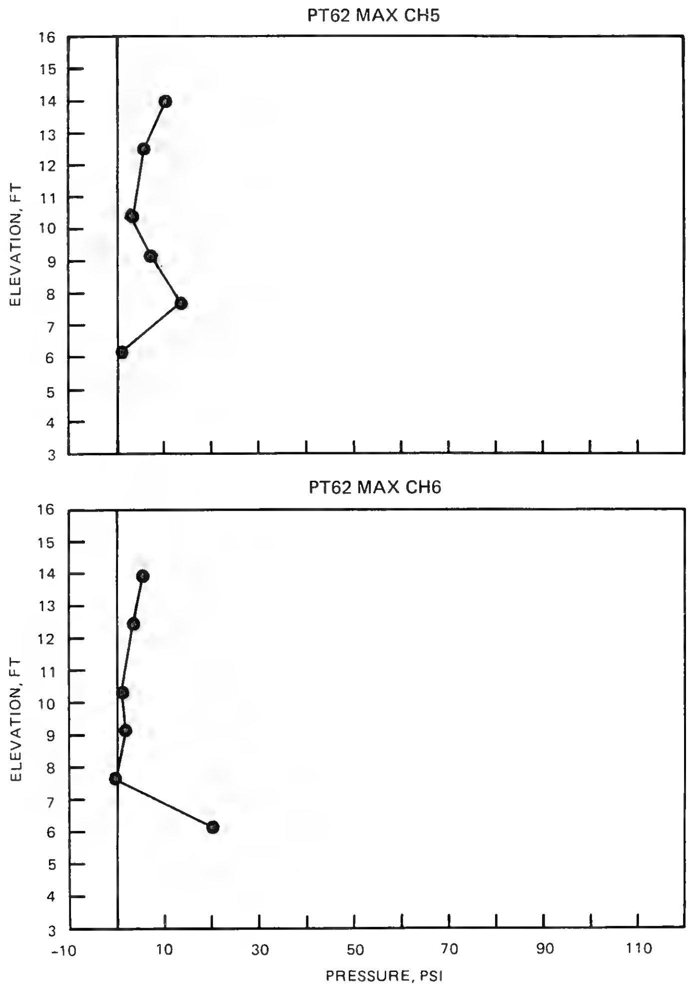


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT62

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

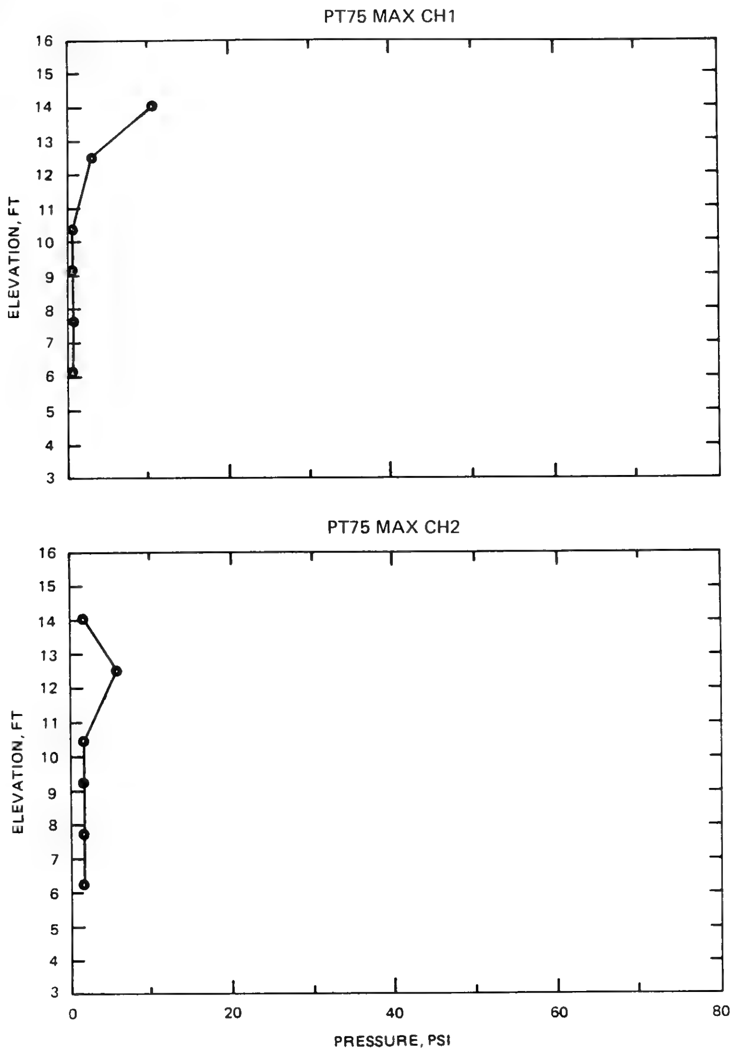


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT62

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

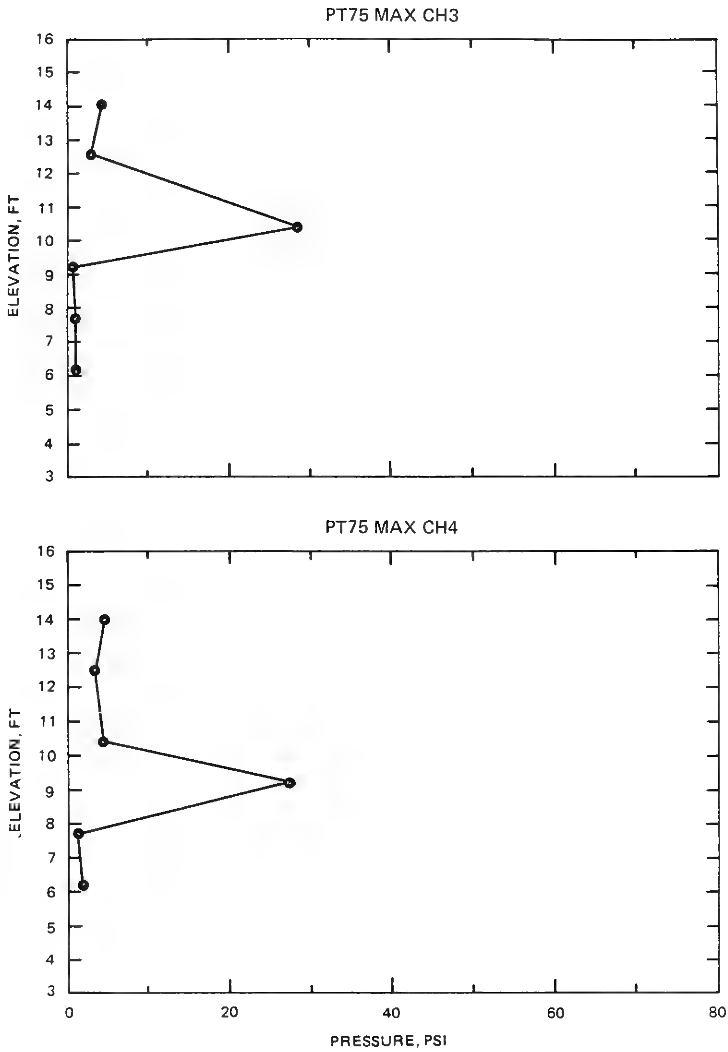


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT75

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

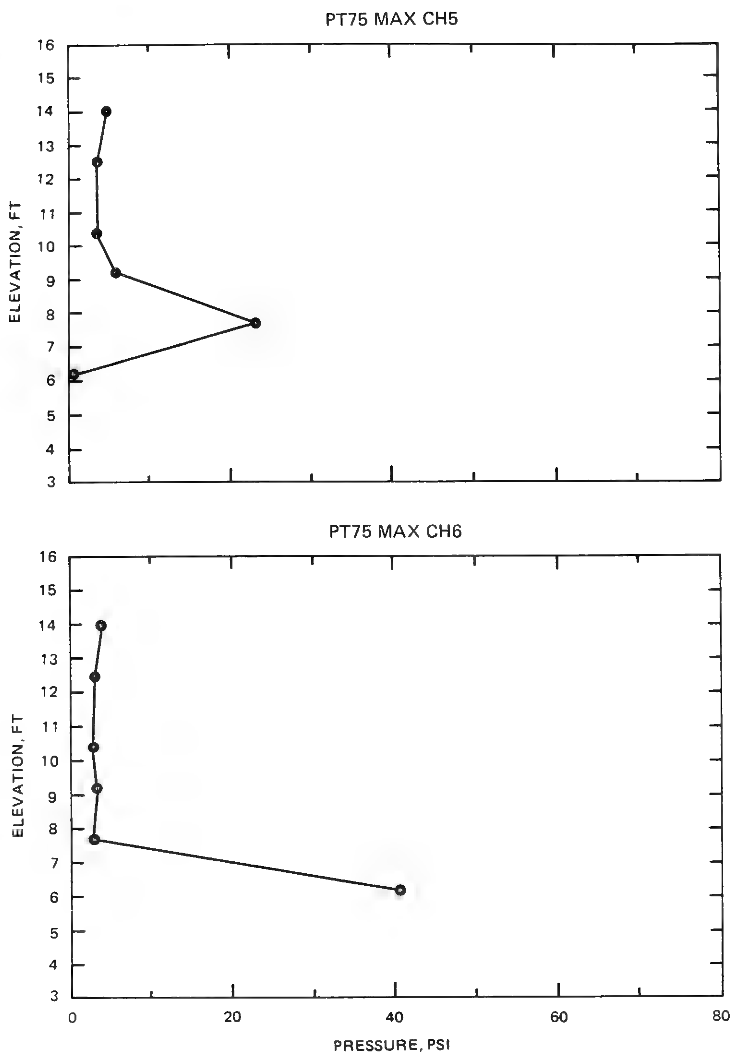


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT75

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

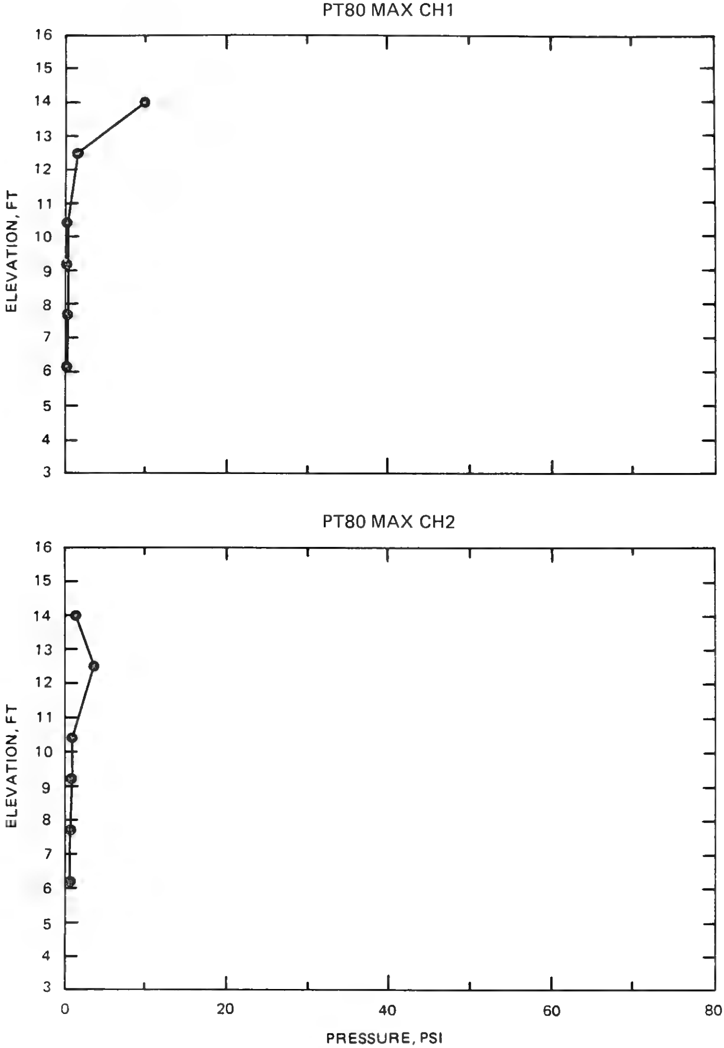


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT75

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

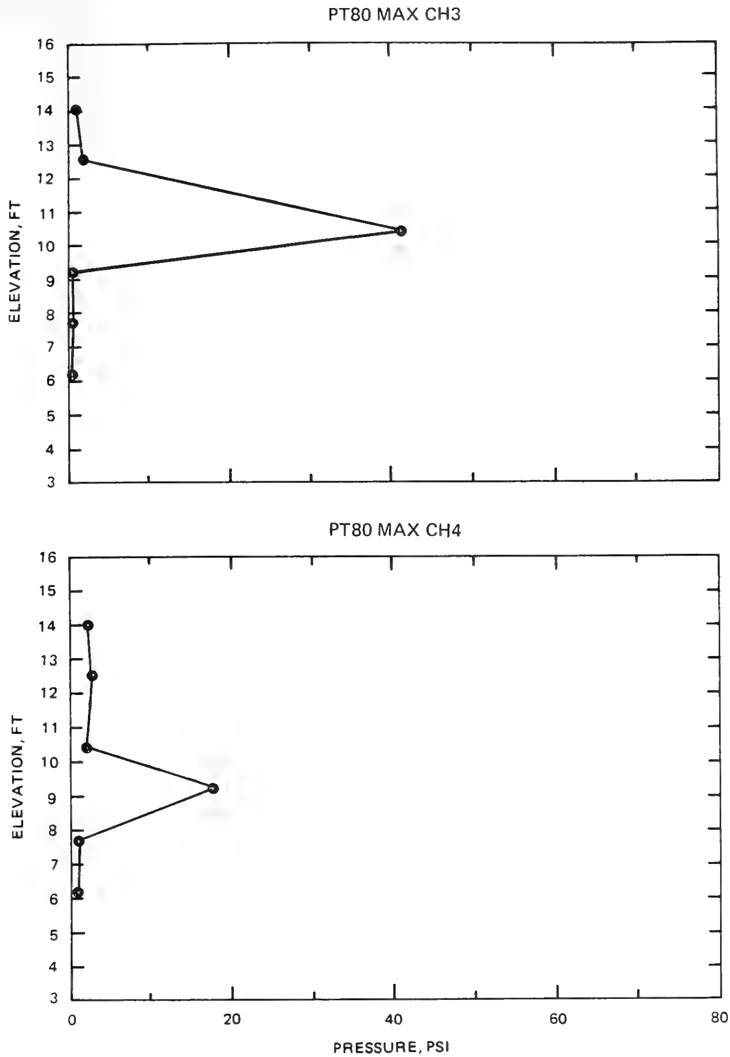


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT80

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

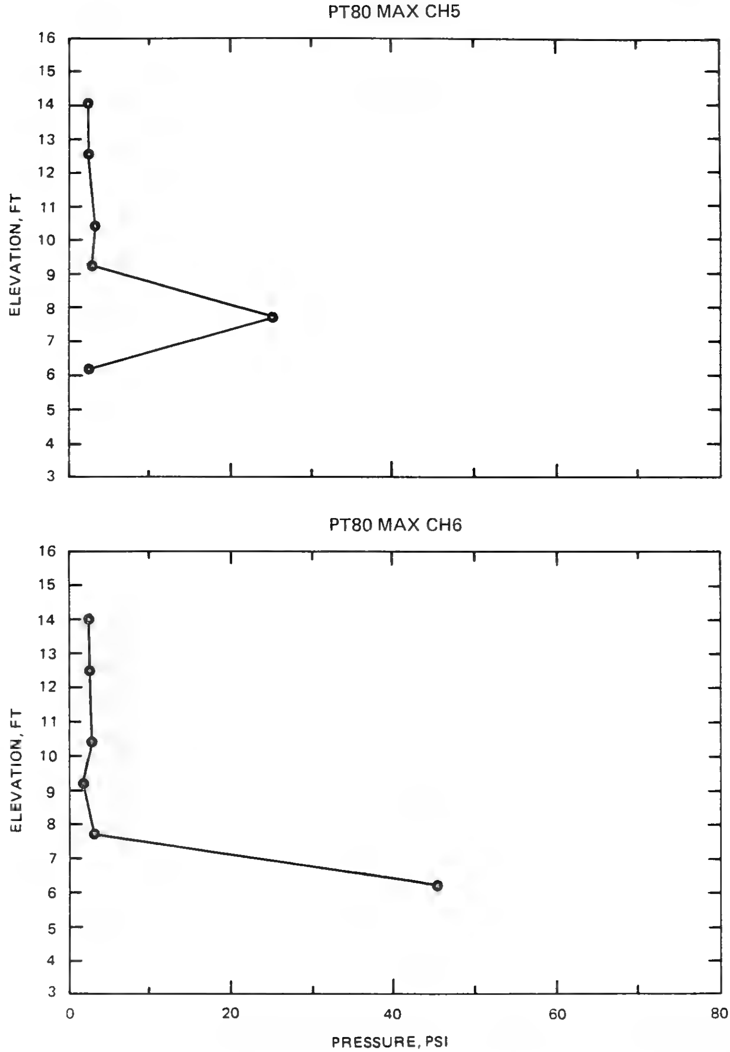


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT80

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

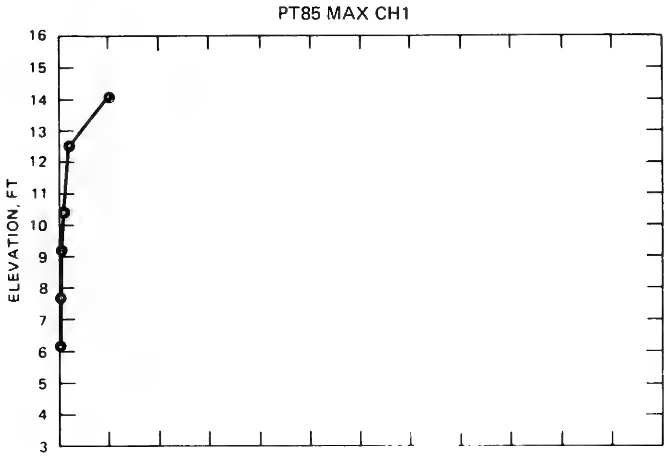


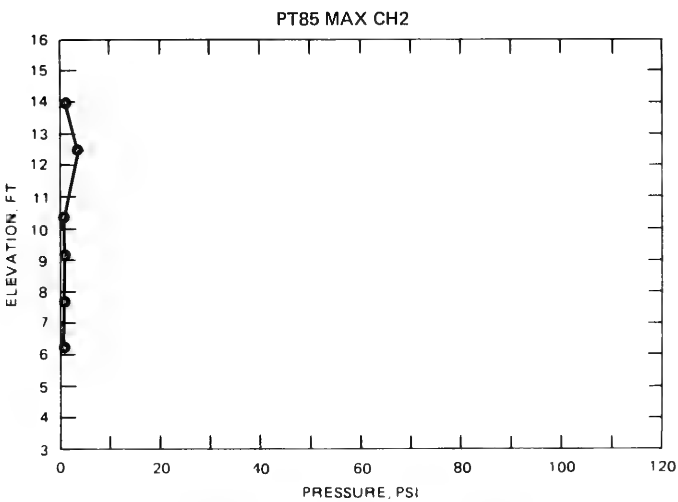
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT80

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6



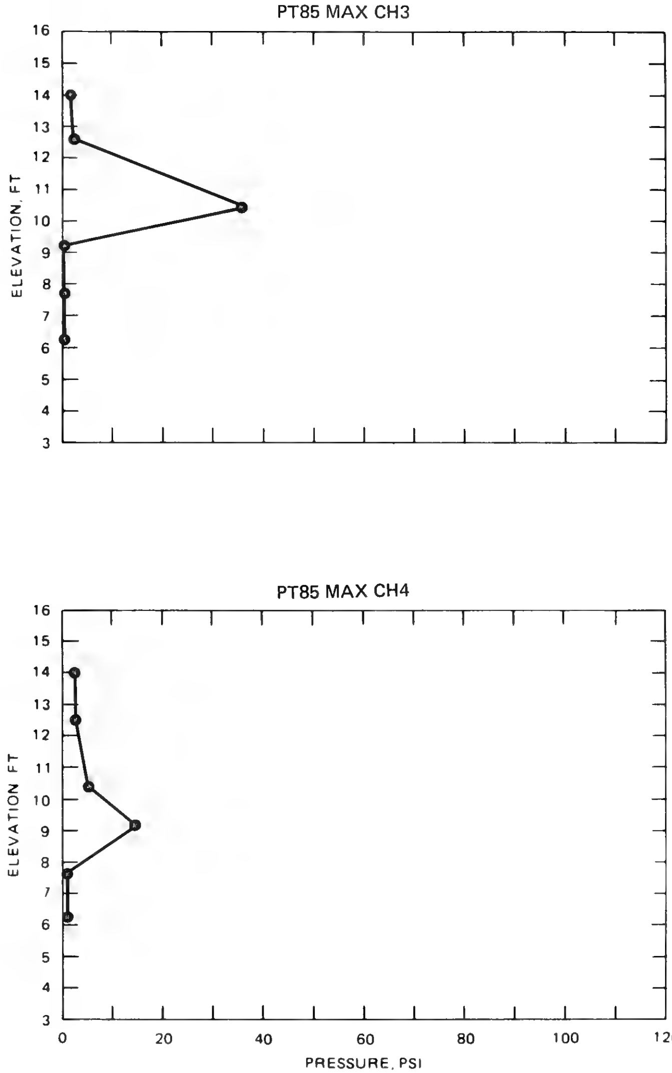


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT85

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

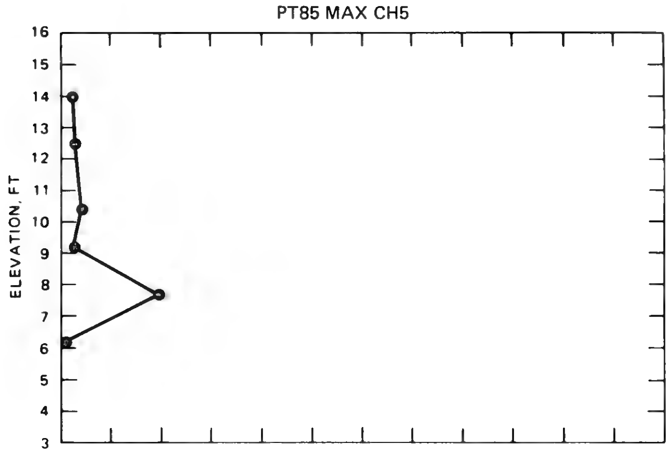


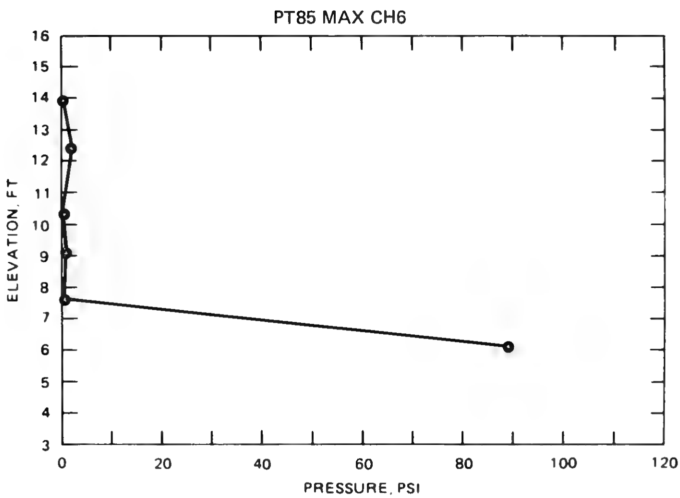
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT85

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4



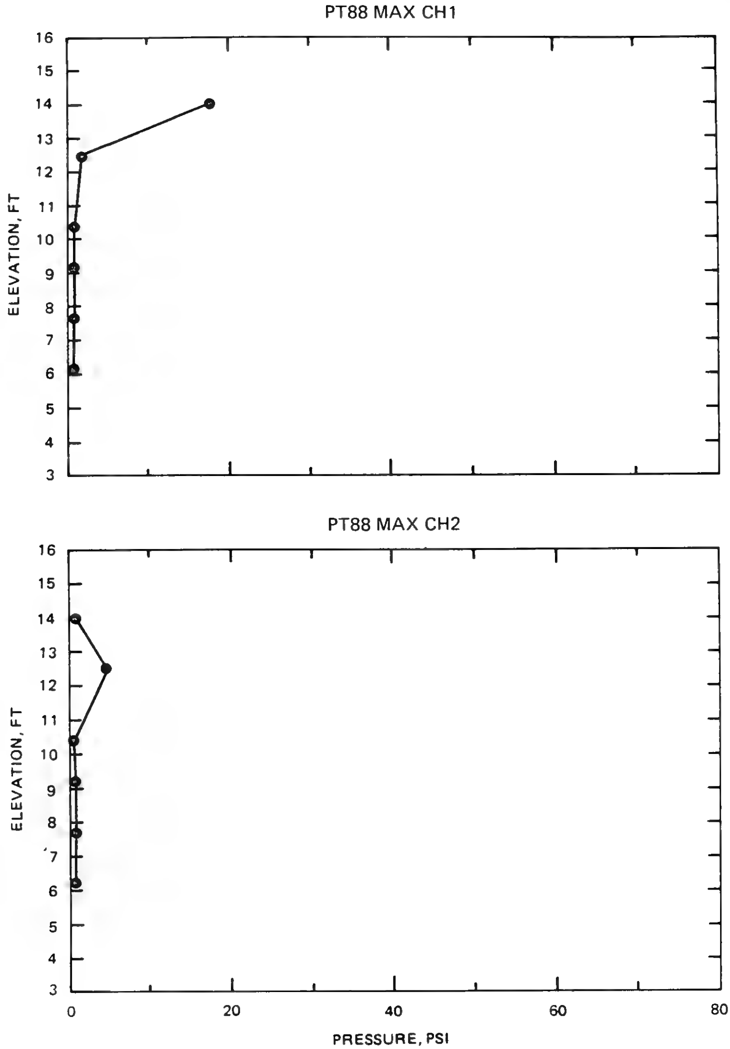


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT85

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

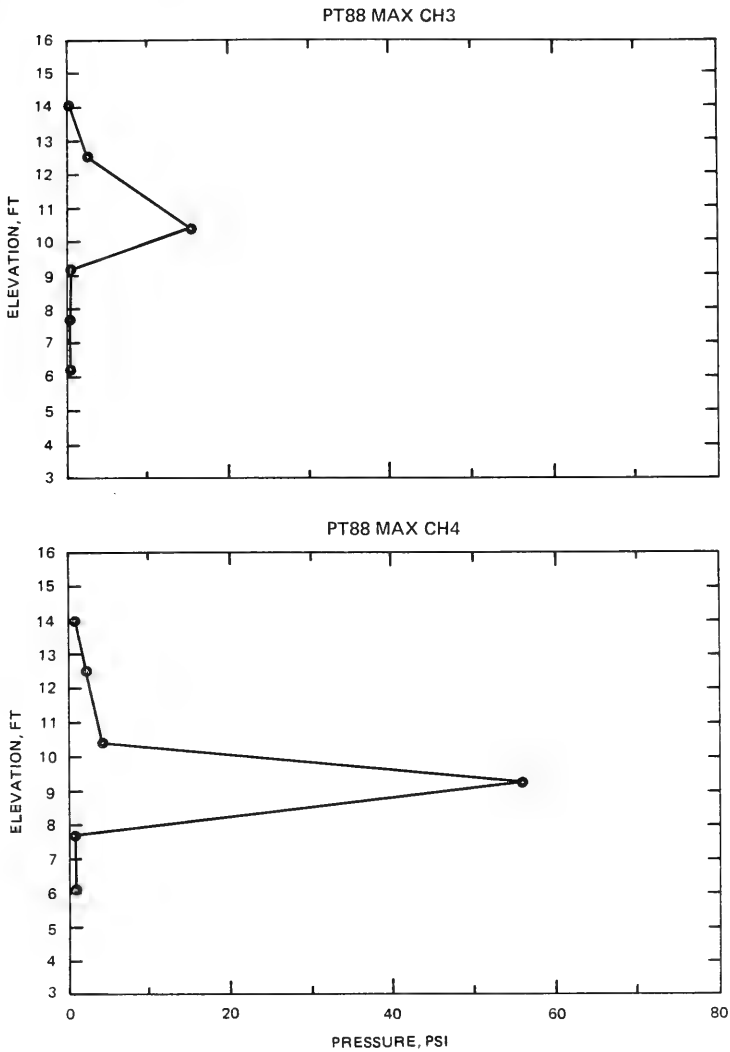


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT88

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

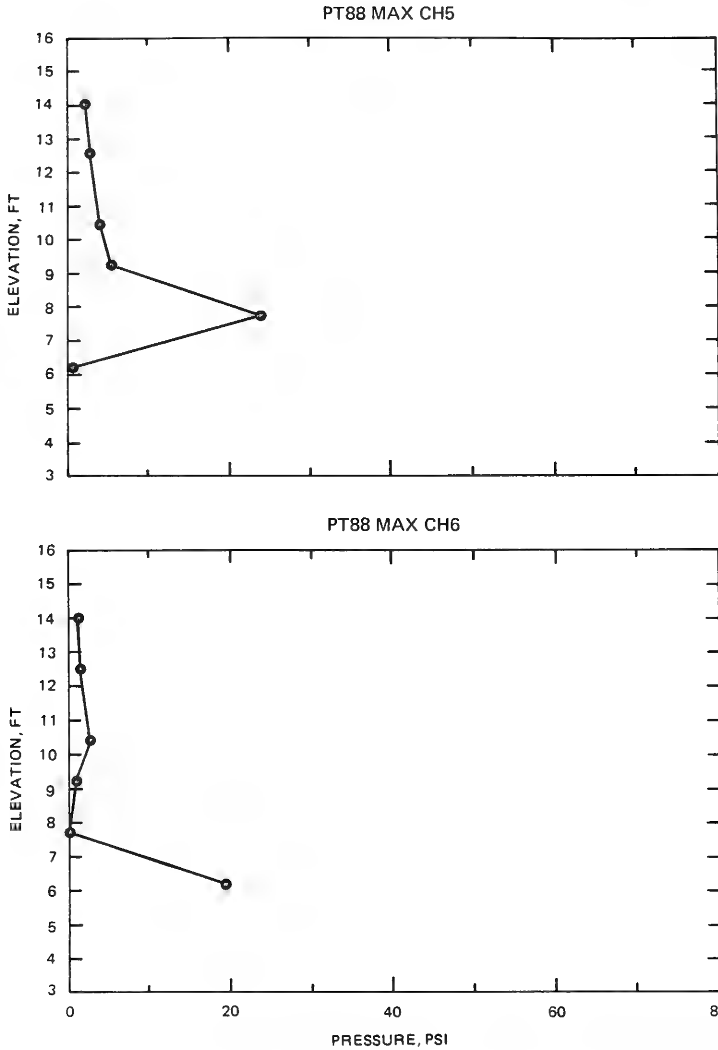


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT88

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

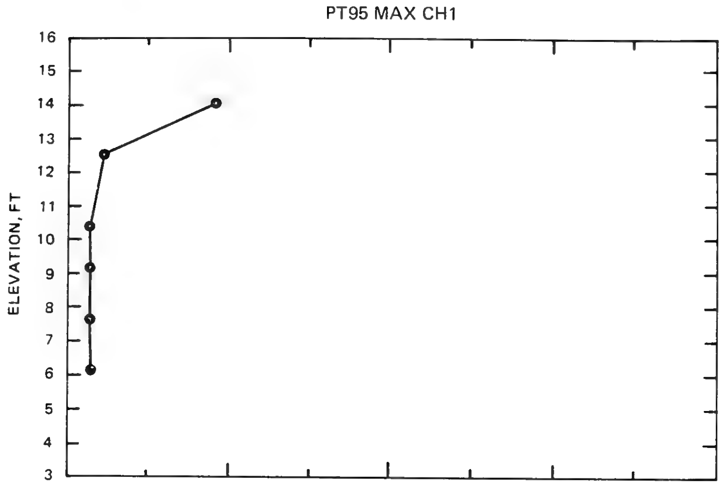


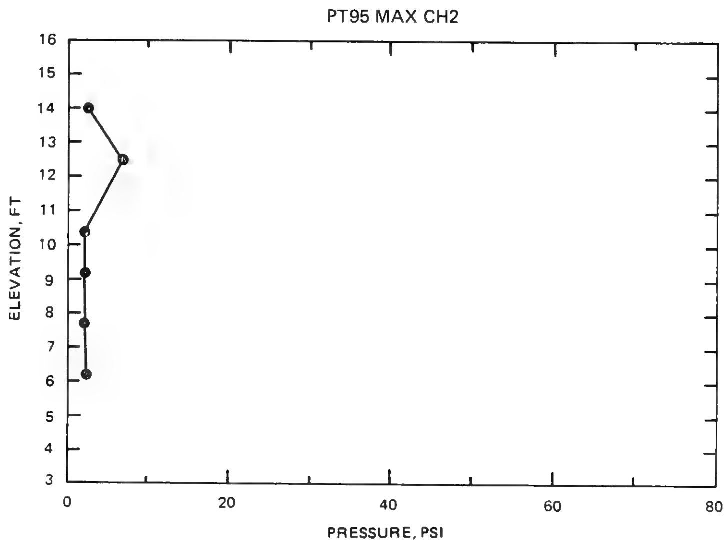
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT88

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6



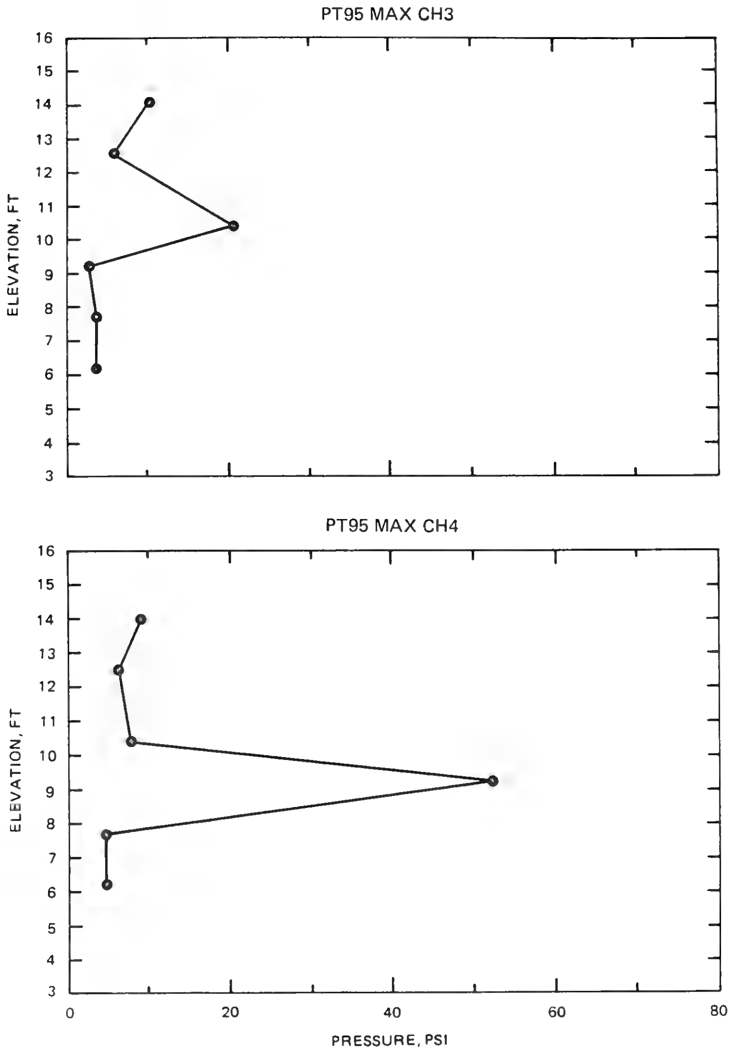


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT95

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

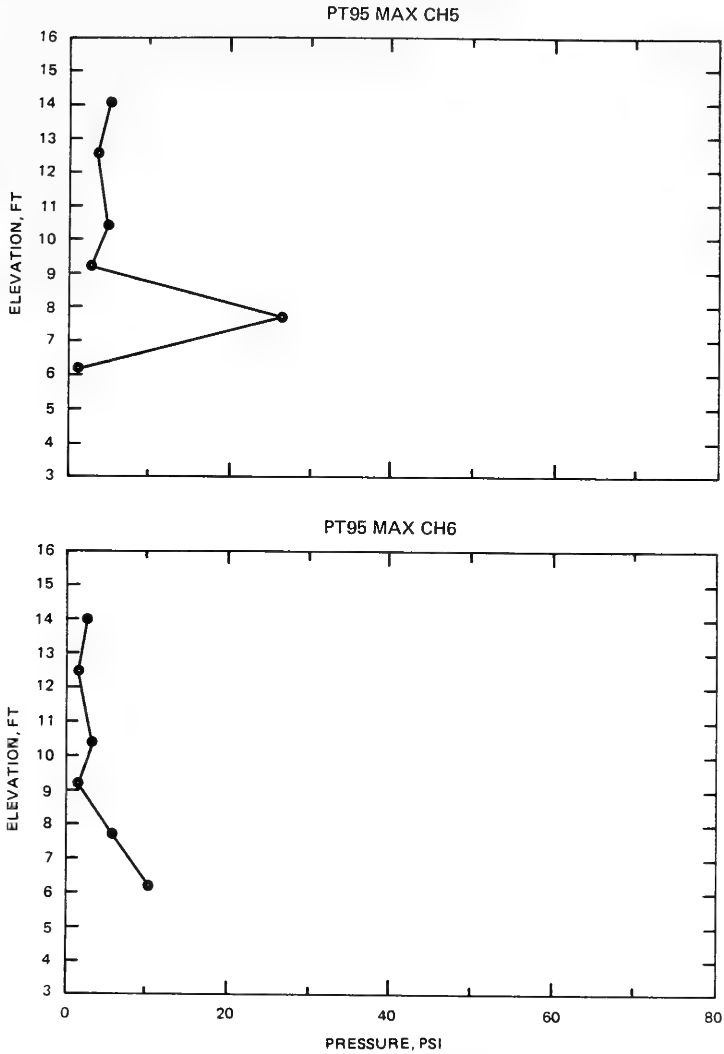


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT95

SWL = +7.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

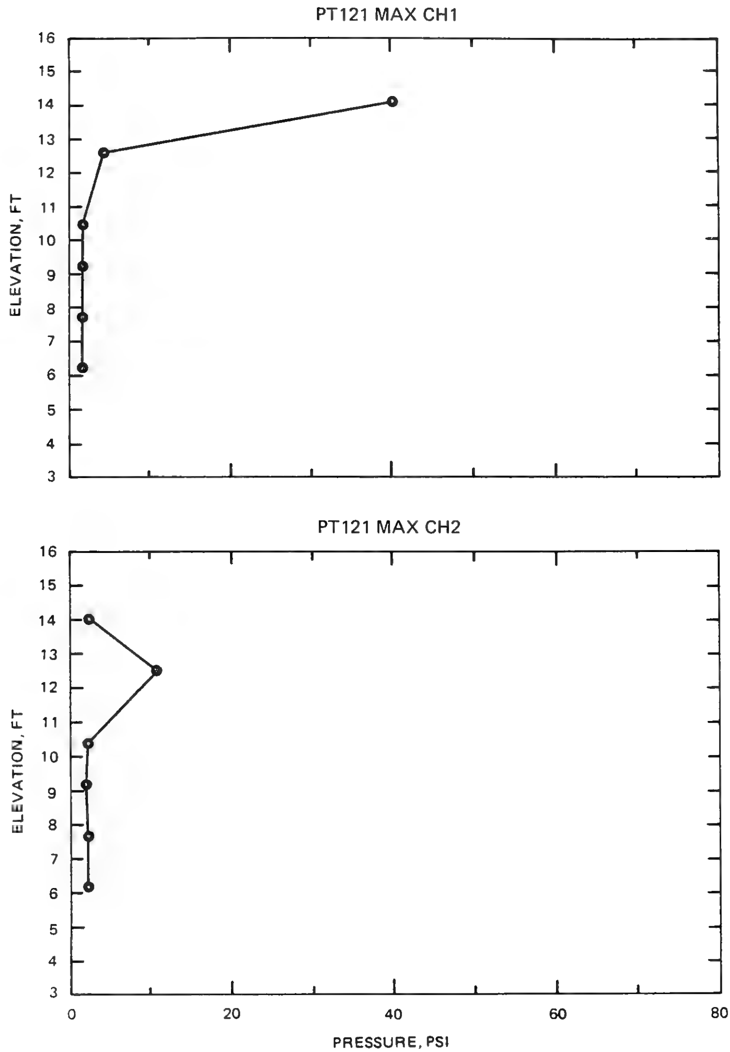


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT95

SWL = +7.0

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

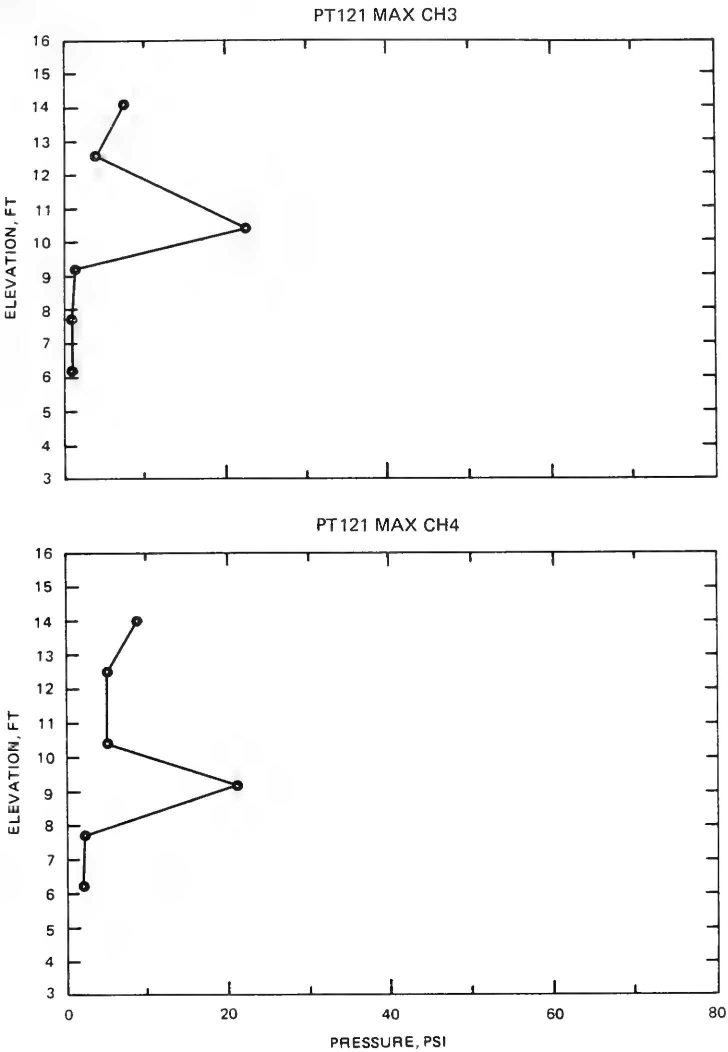


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT121

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

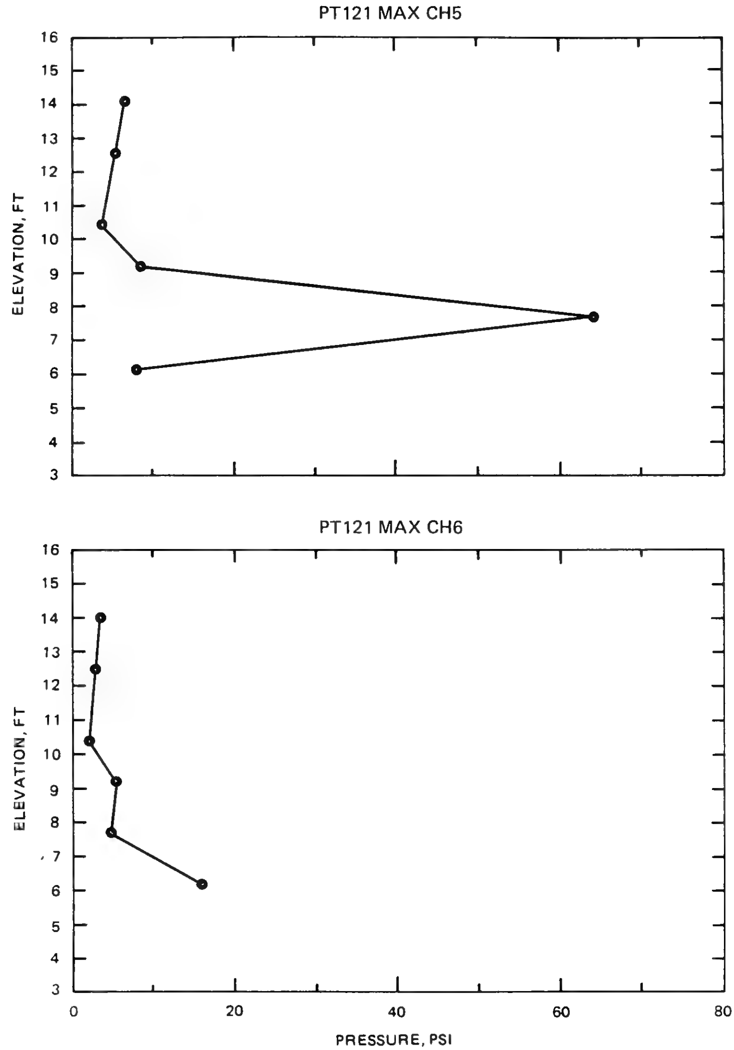


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

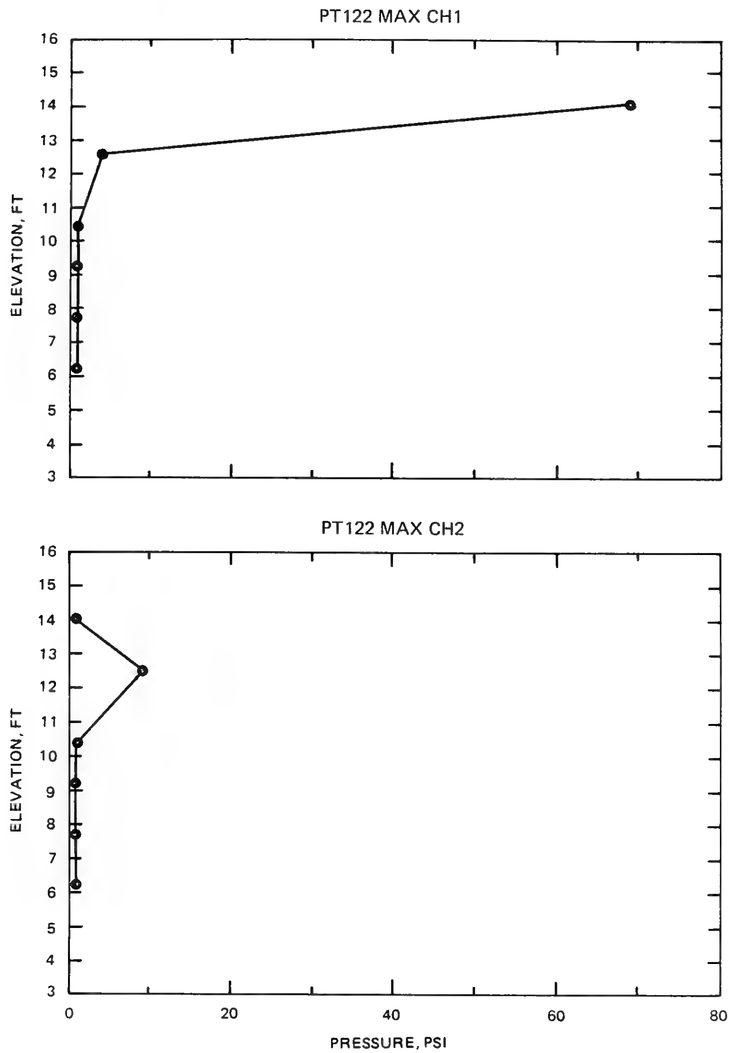
HURRICANE PT121

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4



INSTANTANEOUS WAVE PRESSURE DISTRIBUTION
HURRICANE PT121
SWL = +8.0 FT
MAXIMUM PRESSURES ON CHANNELS 5 AND 6

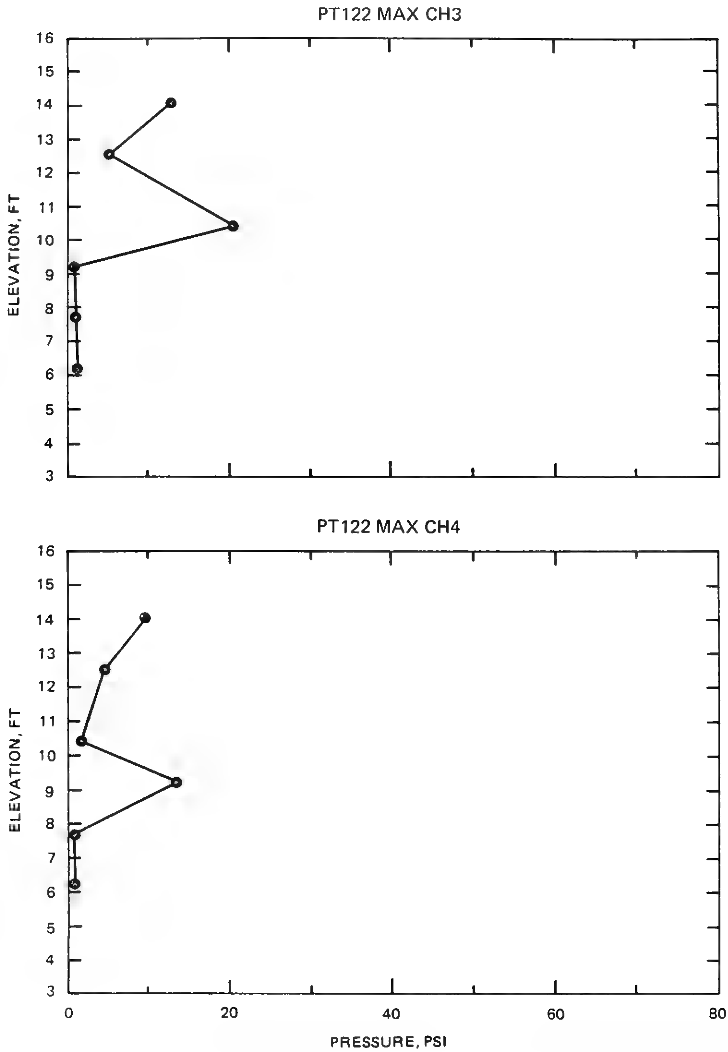


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

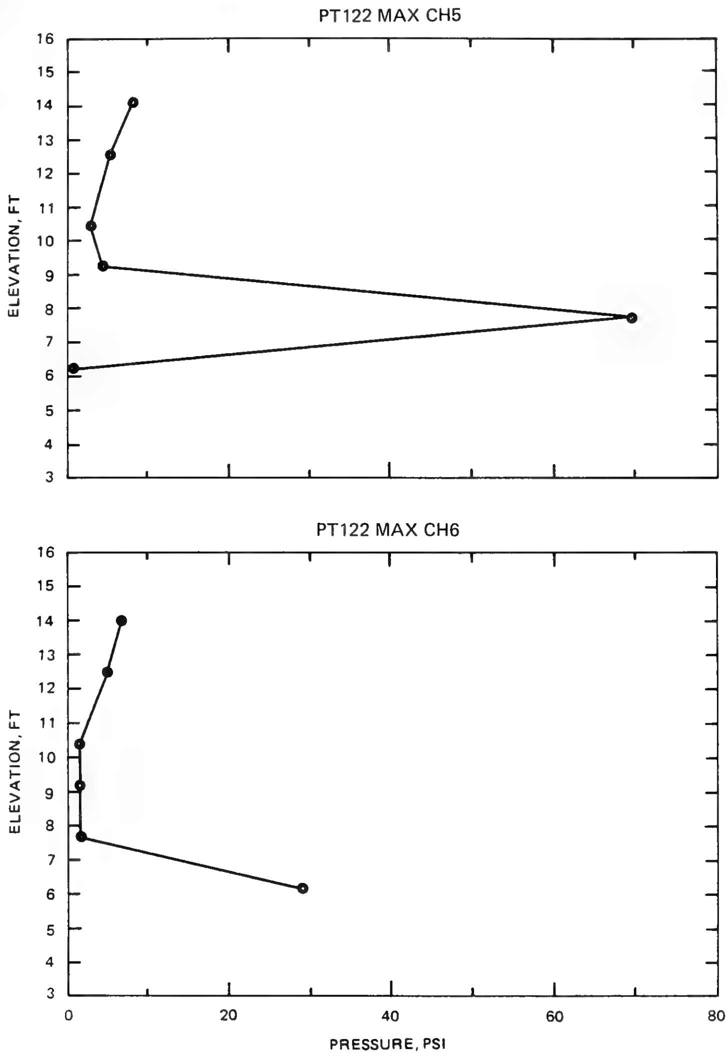
HURRICANE PT122

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2



INSTANTANEOUS WAVE PRESSURE DISTRIBUTION
HURRICANE PT122
SWL = +8.0 FT
MAXIMUM PRESSURES ON CHANNELS 3 AND 4

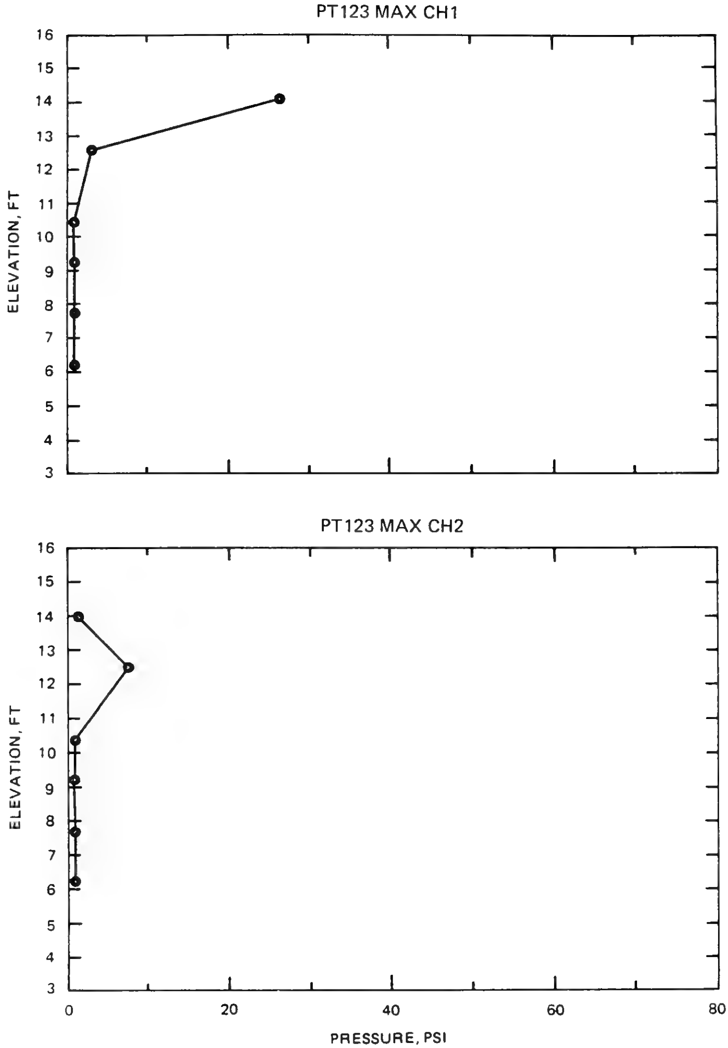


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT122

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

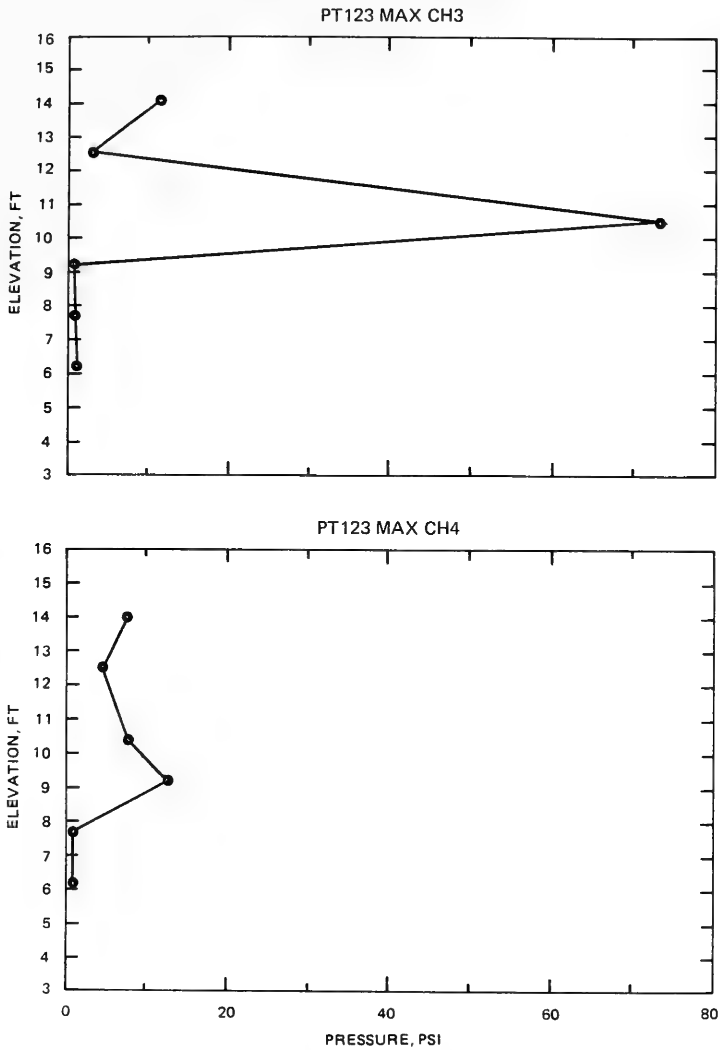


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT123

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

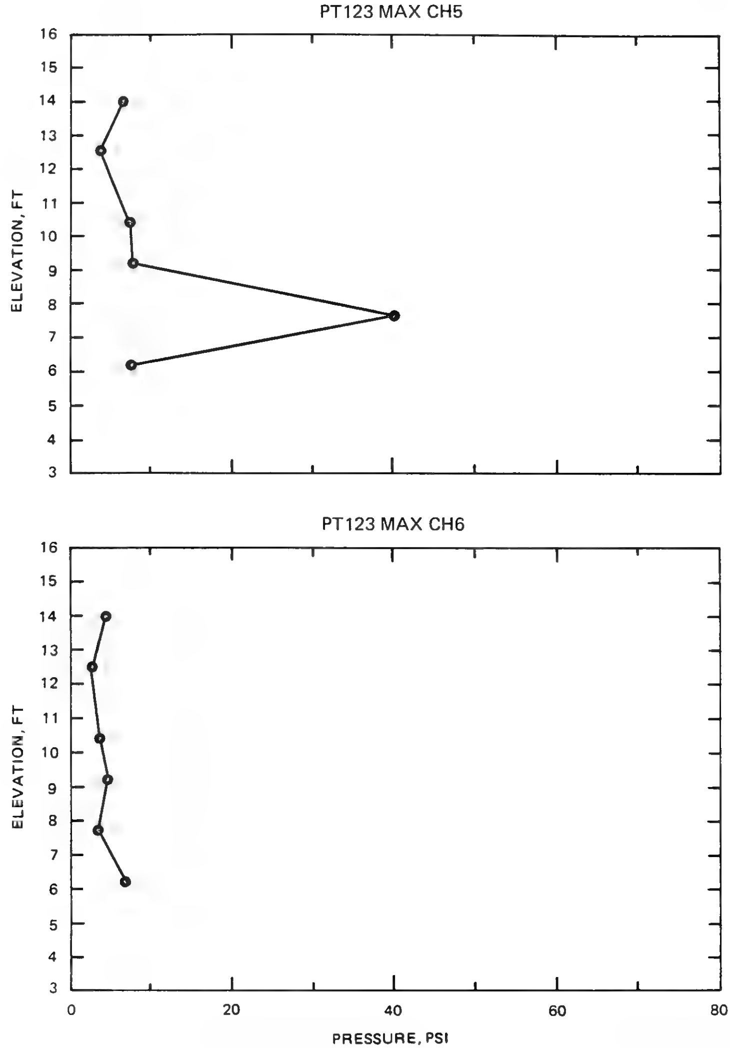


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

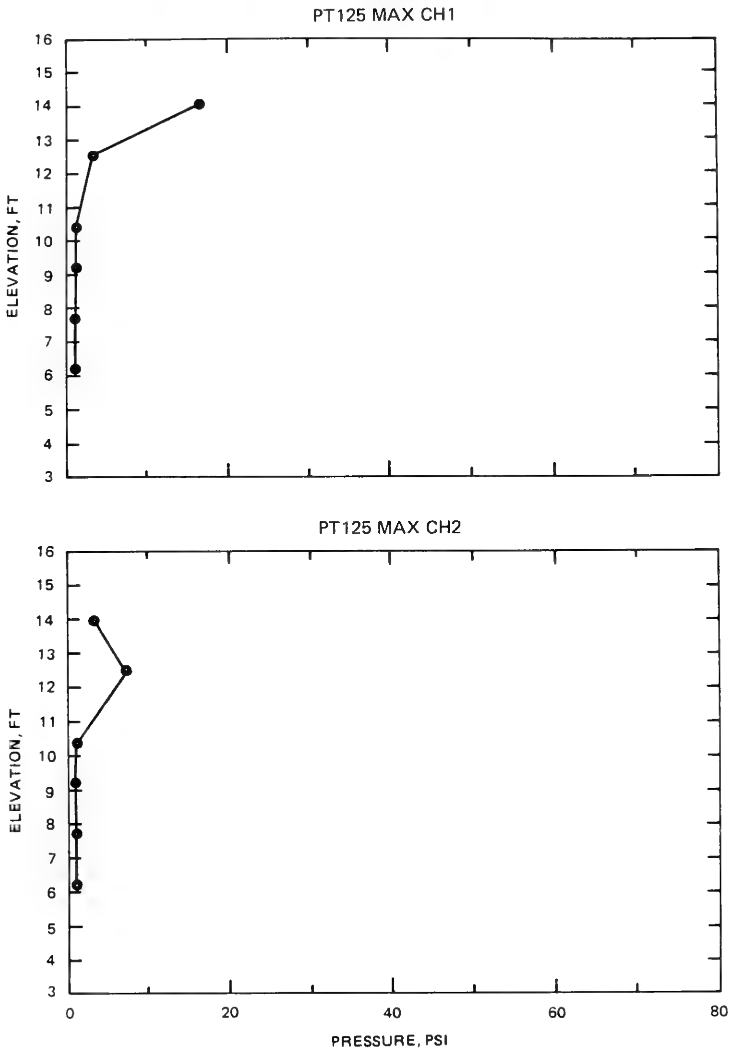
HURRICANE PT123

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4



INSTANTANEOUS WAVE PRESSURE DISTRIBUTION
HURRICANE PT123
SWL = +8.0 FT
MAXIMUM PRESSURES ON CHANNELS 5 AND 6

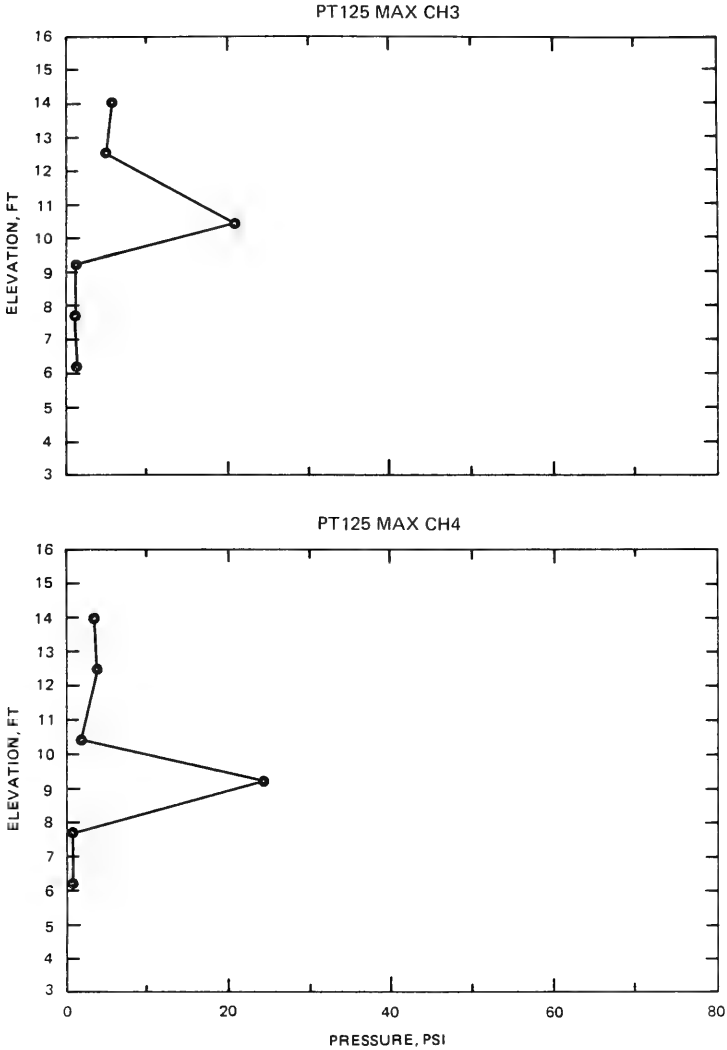


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT125

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

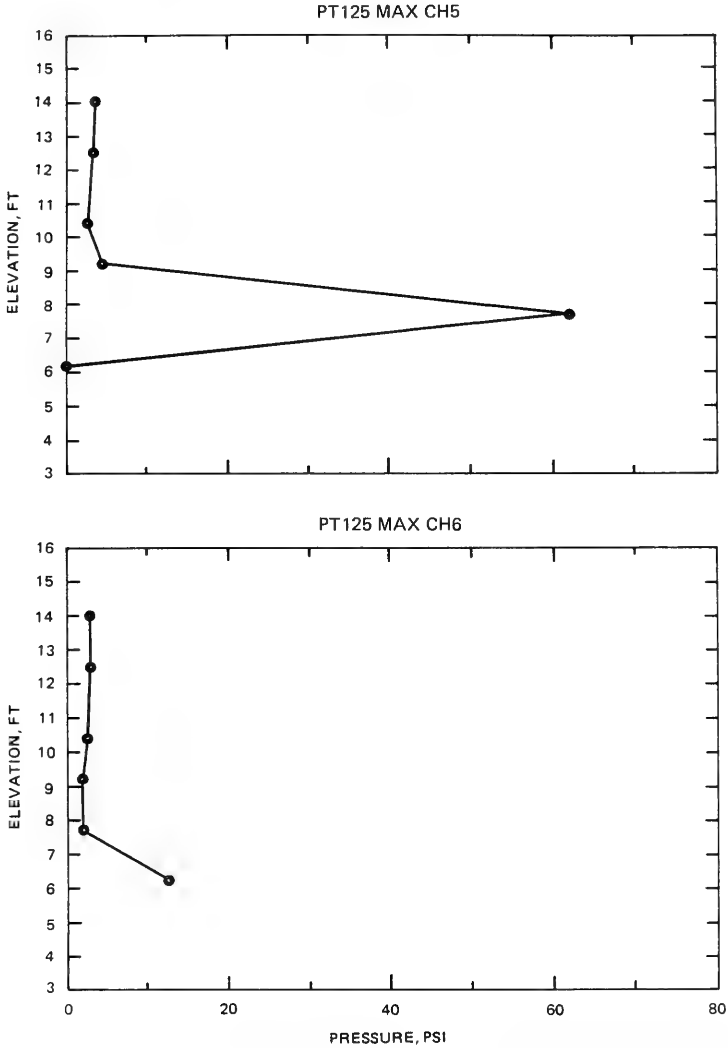


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT125

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

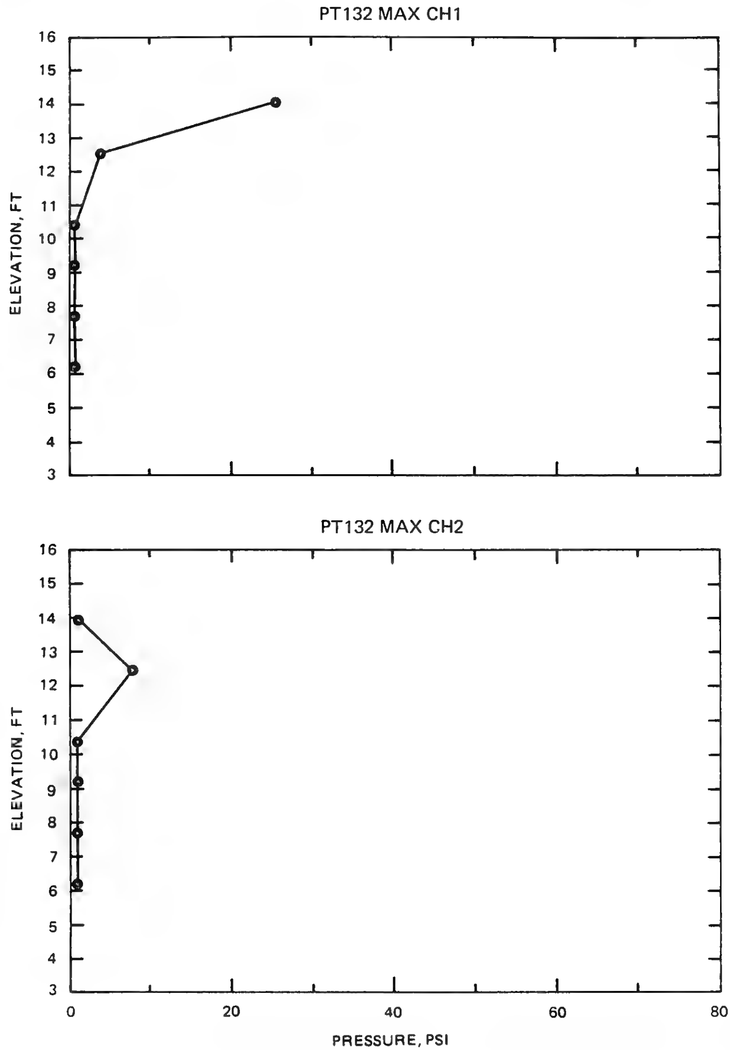


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT125

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

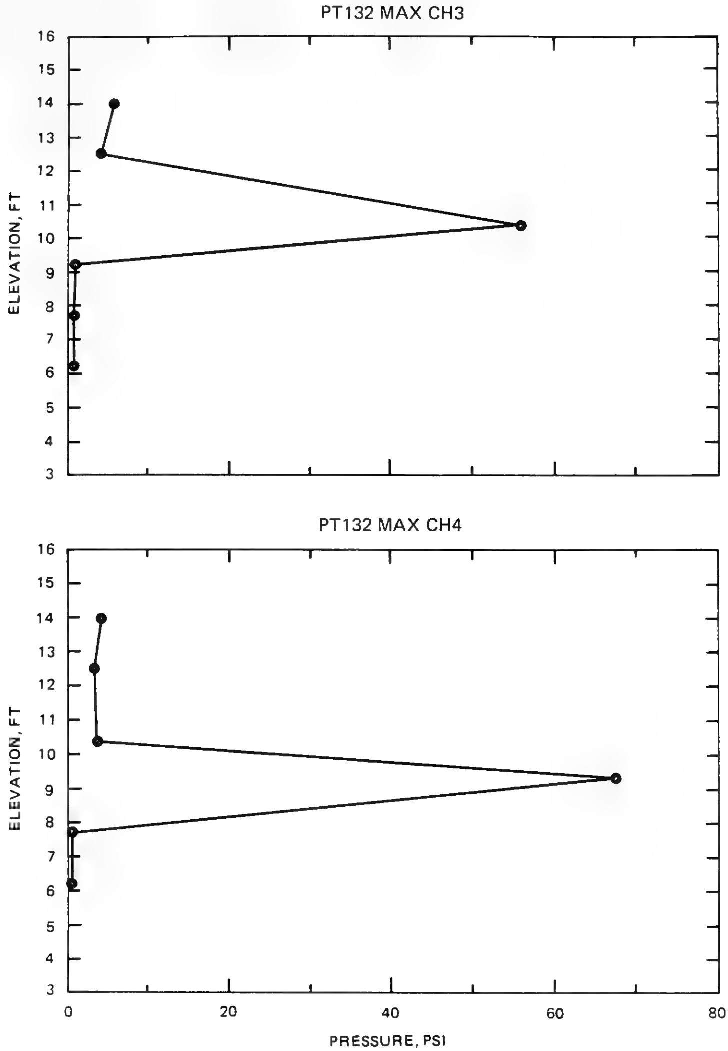


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

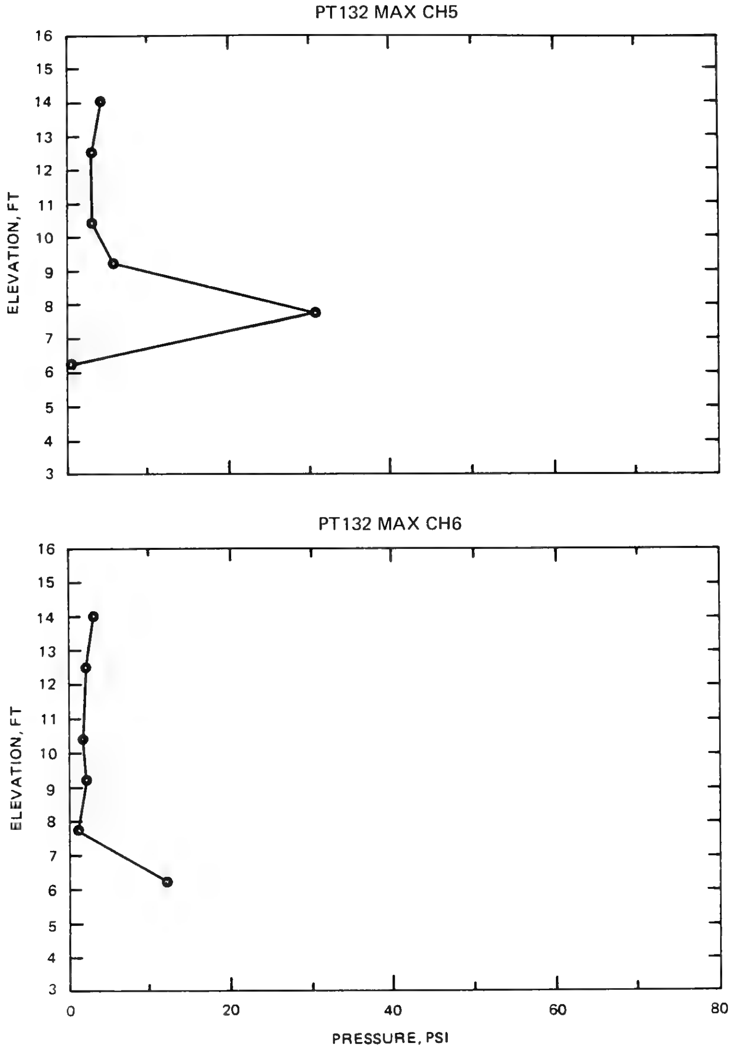
HURRICANE PT132

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2



INSTANTANEOUS WAVE PRESSURE DISTRIBUTION
HURRICANE PT132
SWL = +9.5 FT
MAXIMUM PRESSURES ON CHANNELS 3 AND 4

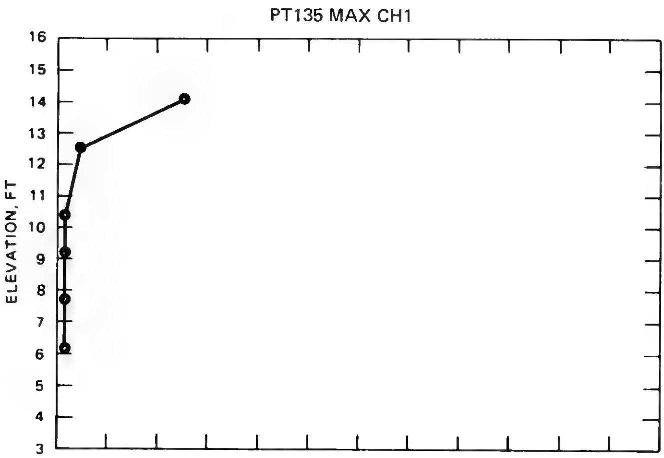


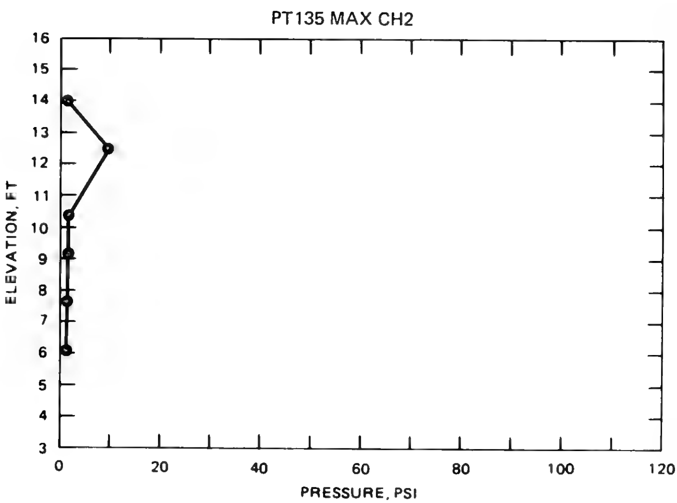
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT132

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6



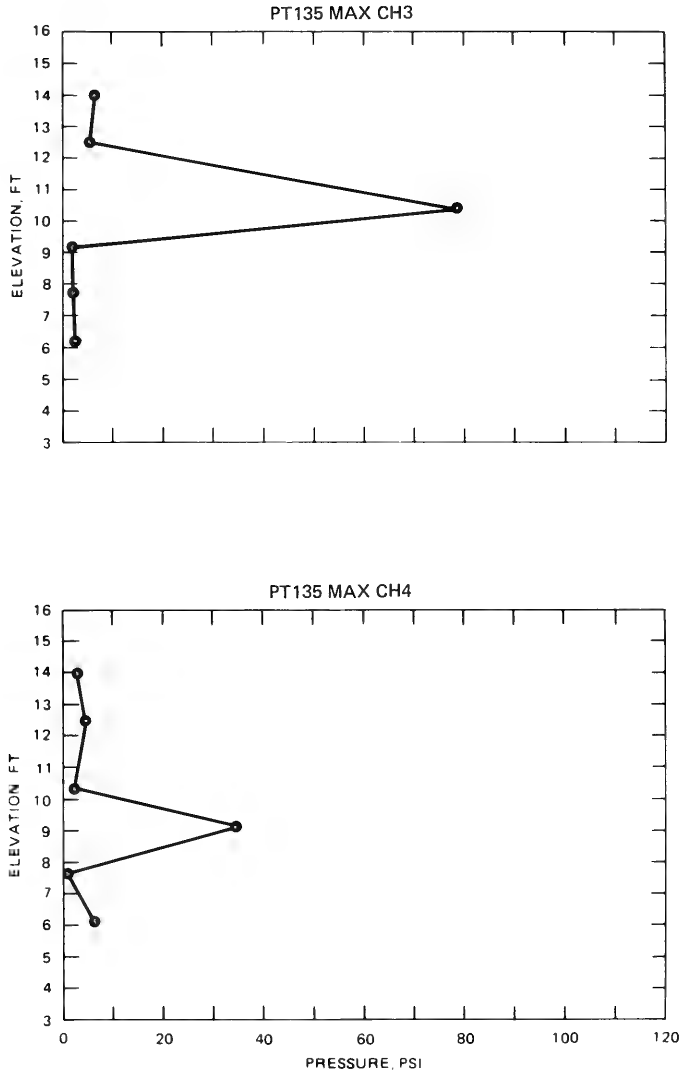


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT135

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

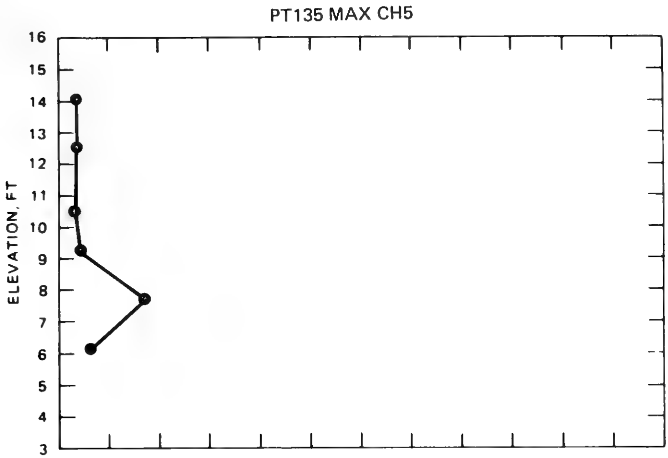


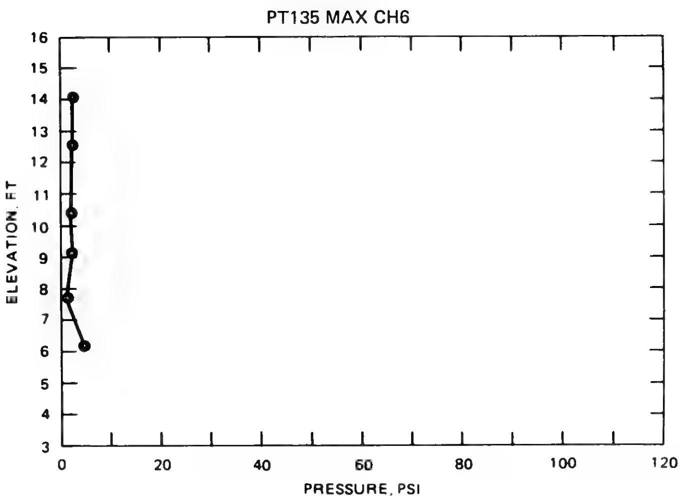
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT135

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4



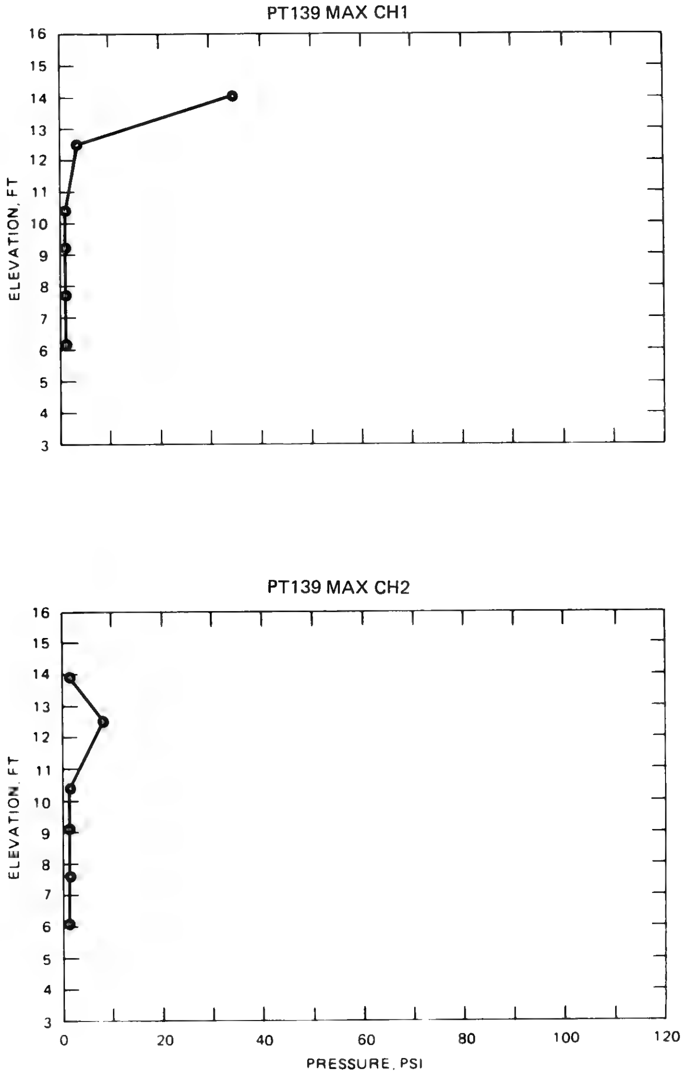


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT135

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

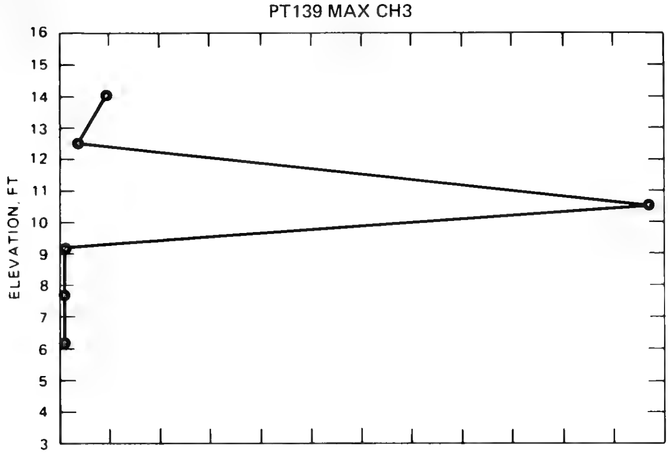


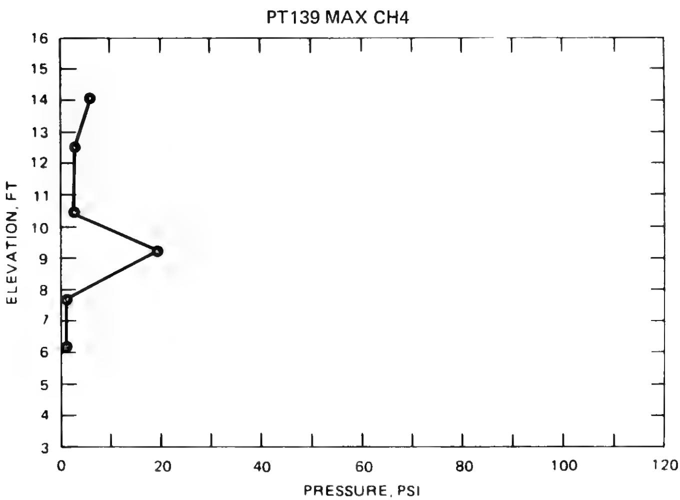
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT139

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2



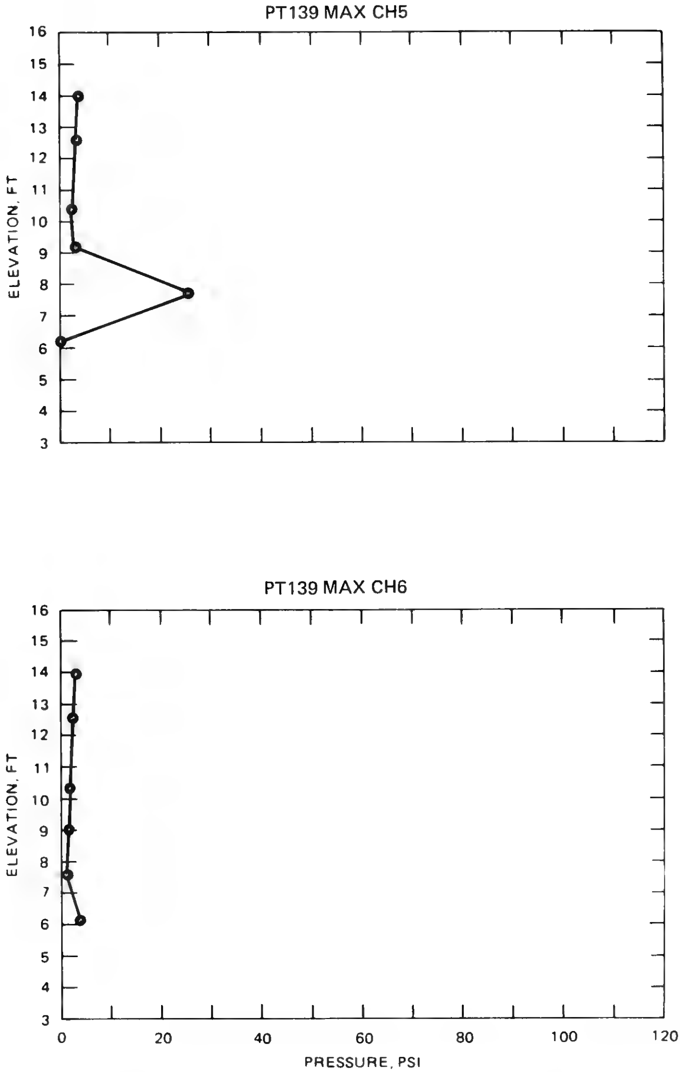


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT139

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

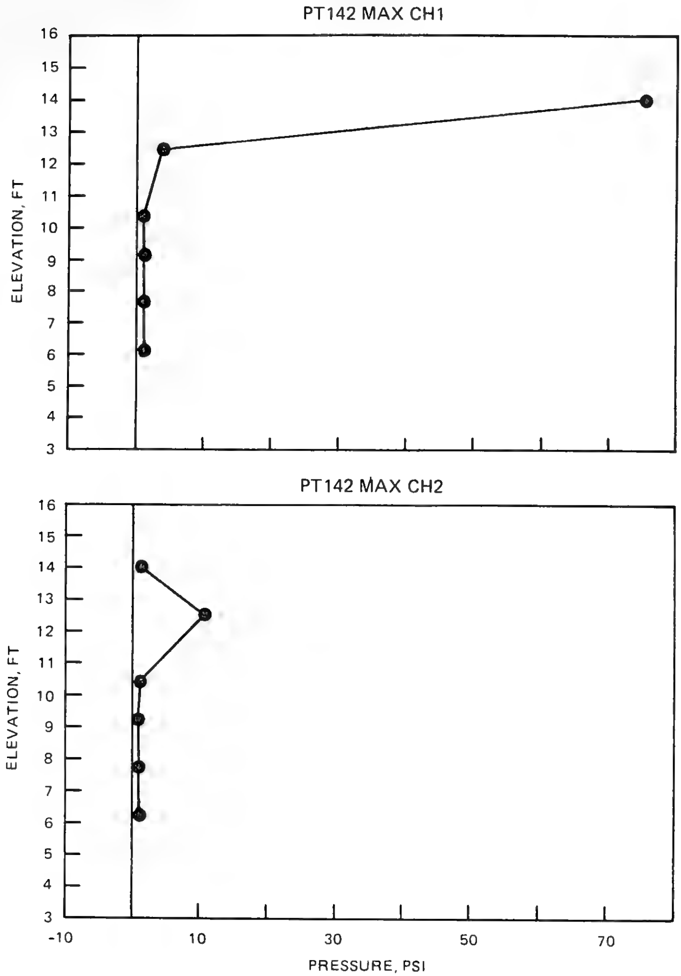


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT139

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

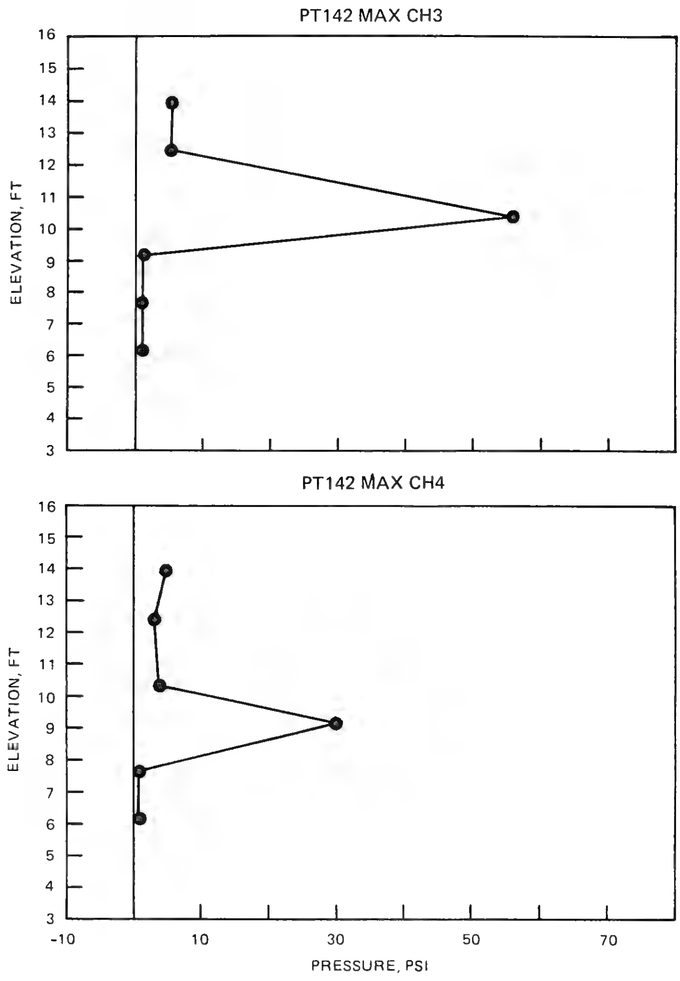


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT142

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

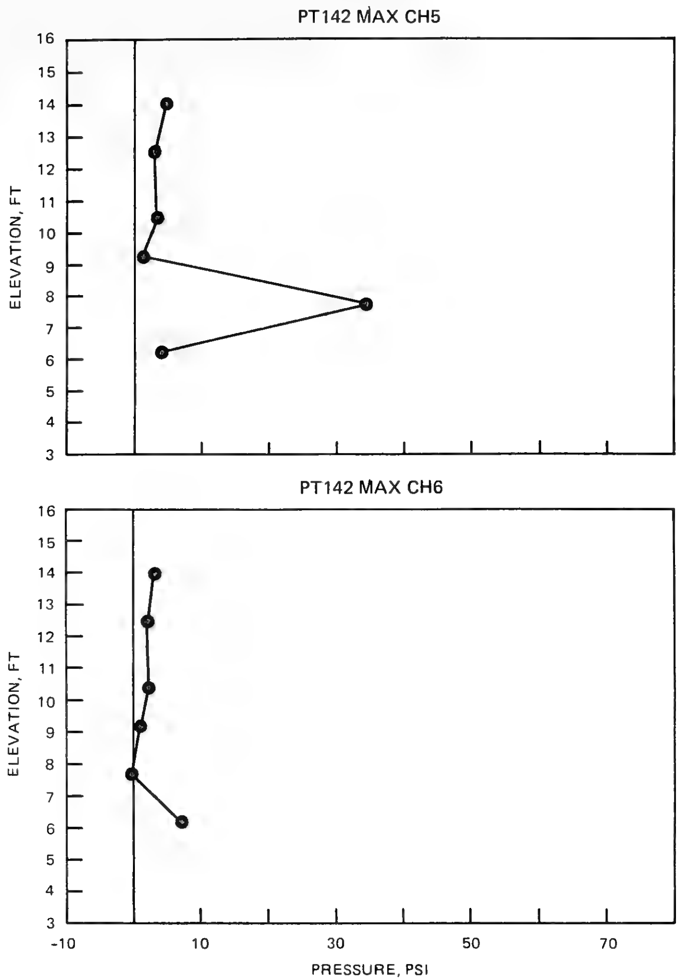


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT142

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

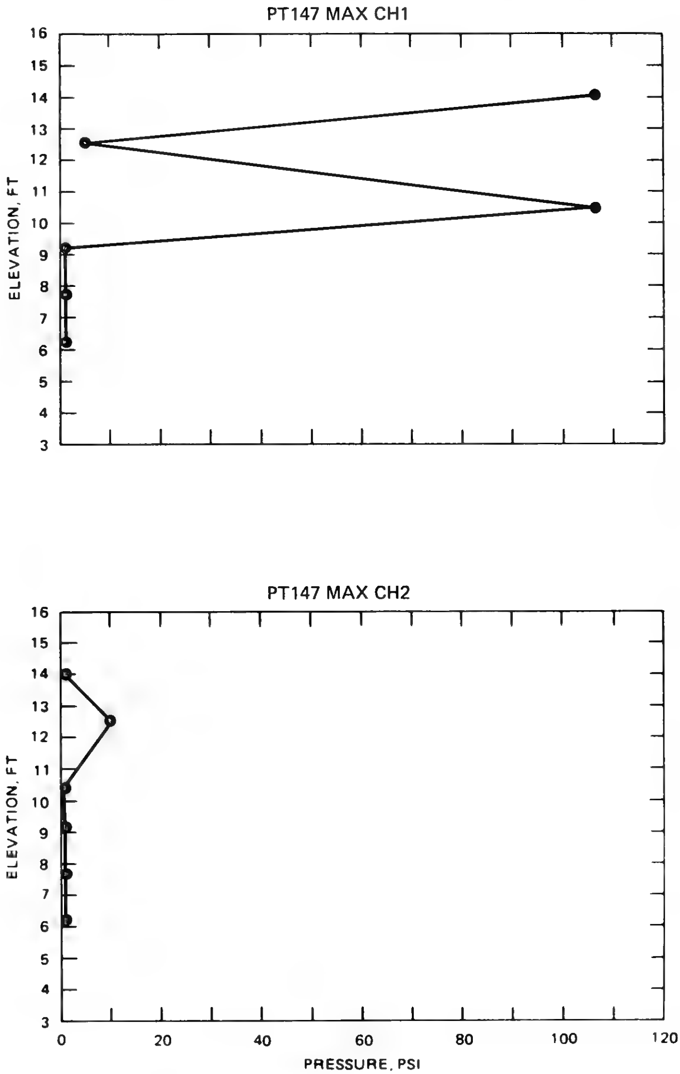


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT142

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

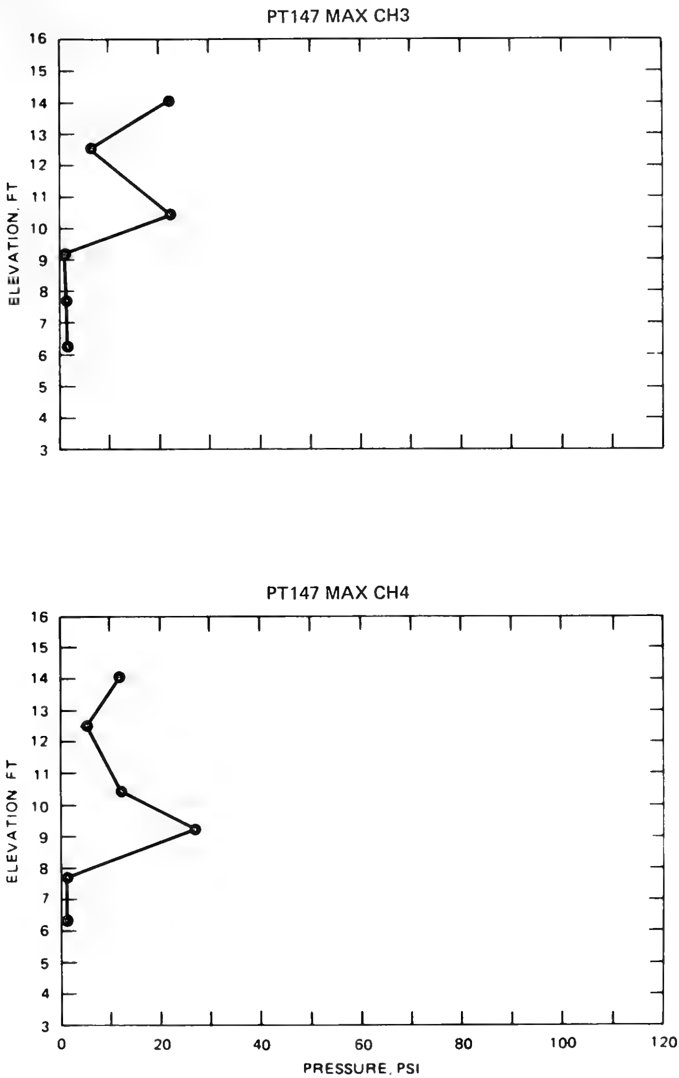


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT147

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

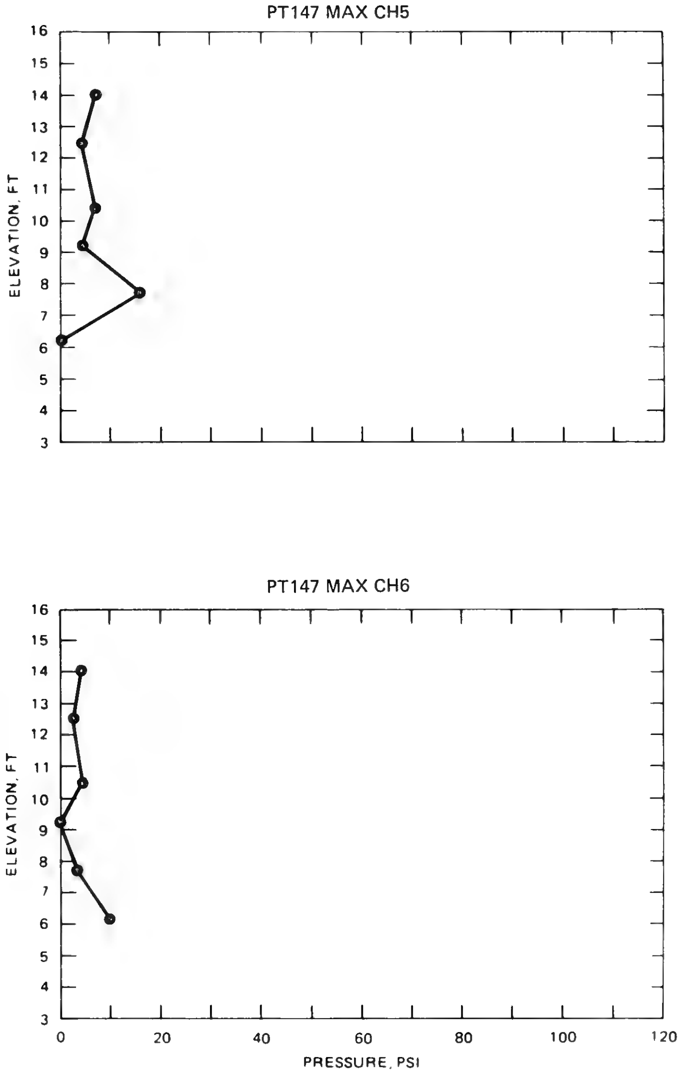


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

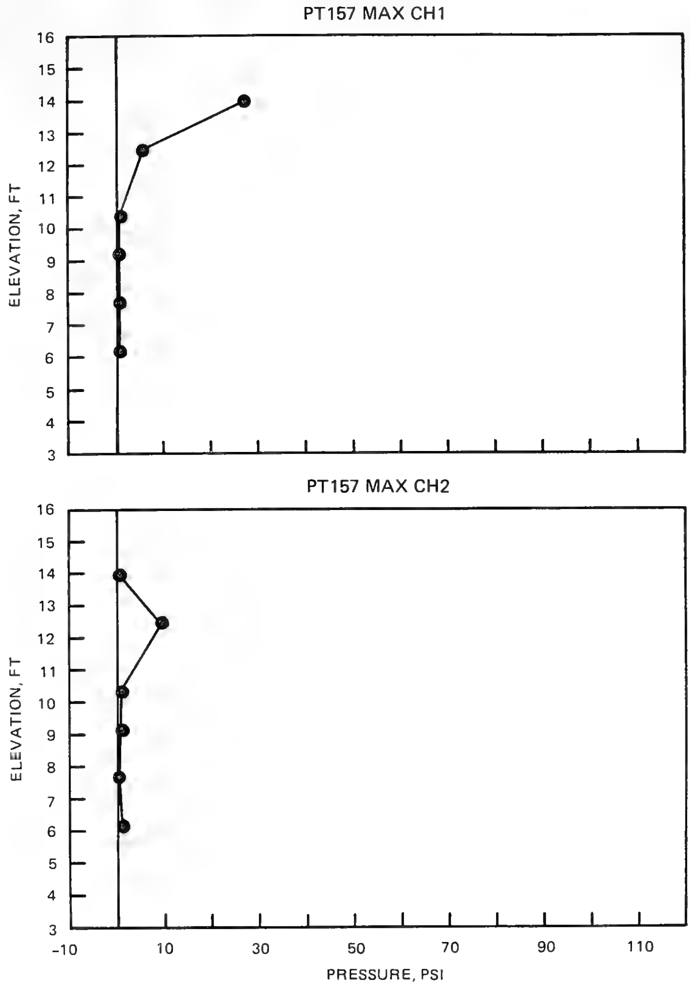
HURRICANE PT147

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4



INSTANTANEOUS WAVE PRESSURE DISTRIBUTION
HURRICANE PT147
SWL = +9.5 FT
MAXIMUM PRESSURES ON CHANNELS 5 AND 6

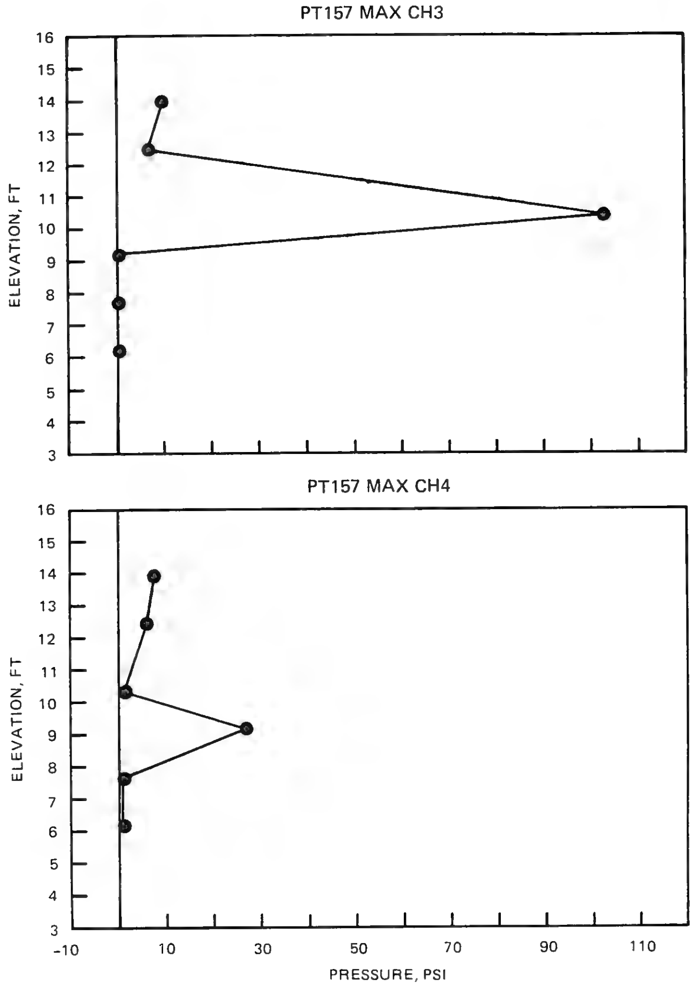


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT157

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2

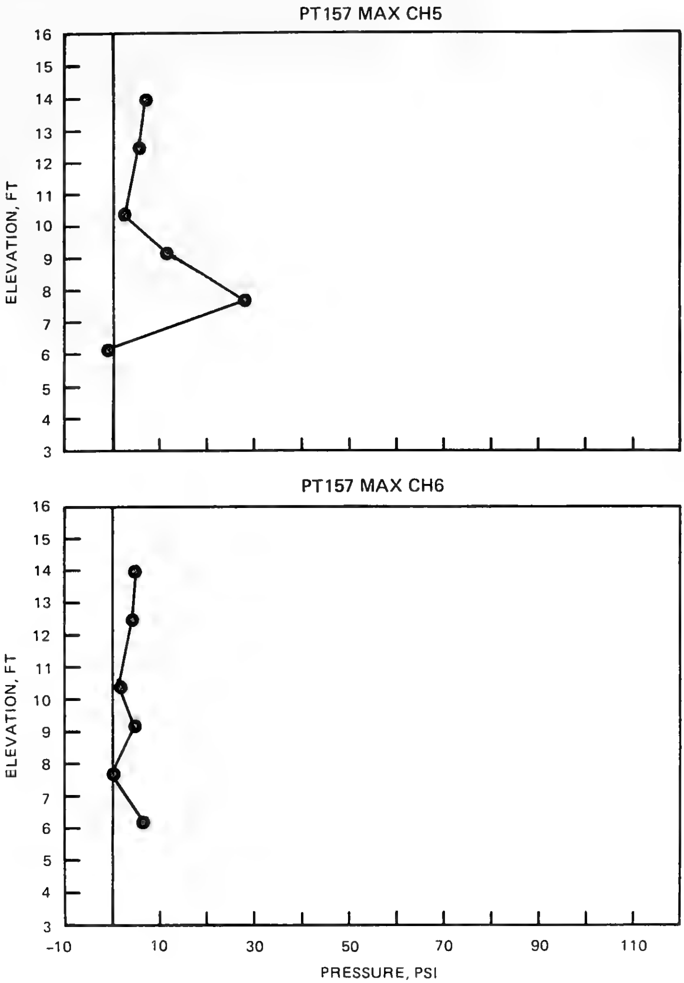


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

HURRICANE PT157

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

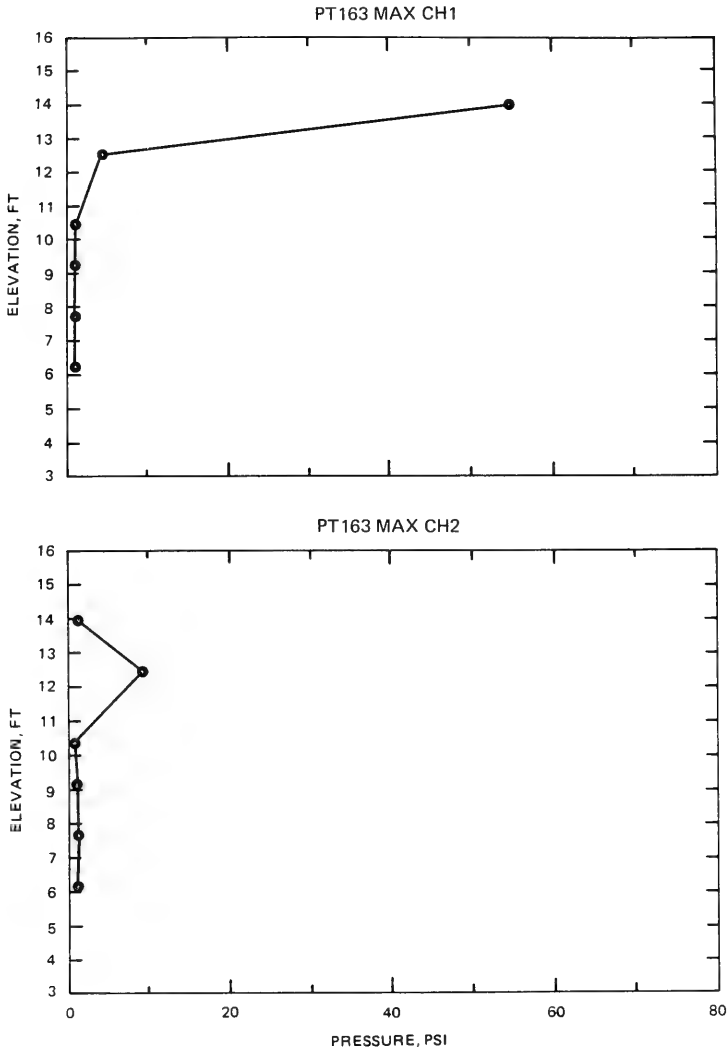


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

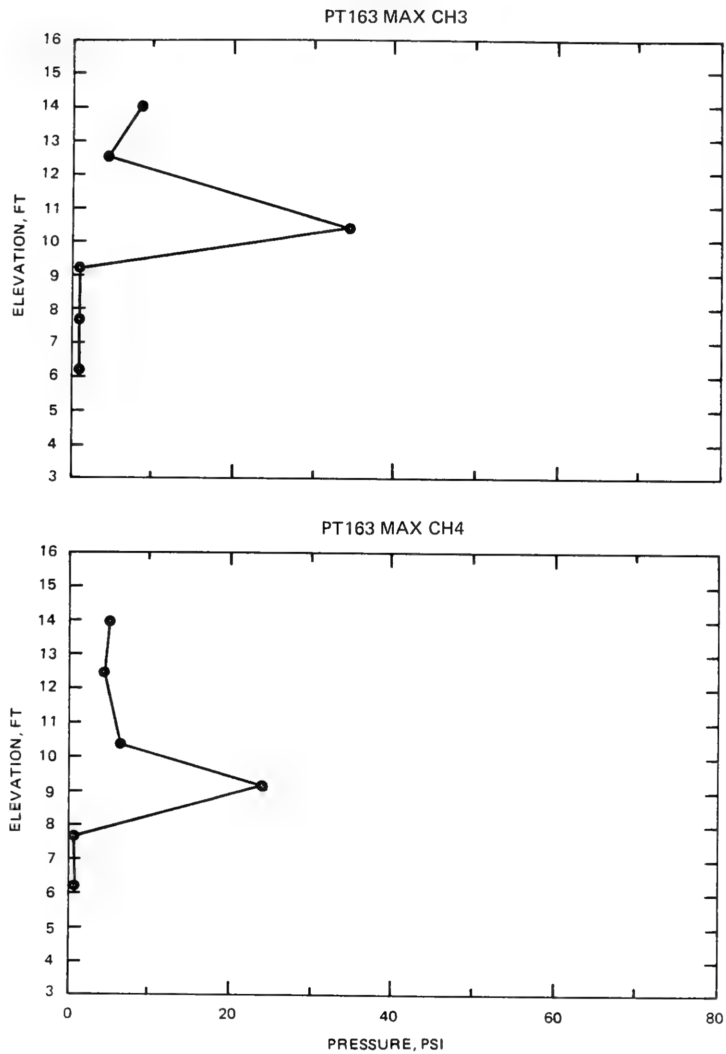
HURRICANE PT157

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6



INSTANTANEOUS WAVE PRESSURE DISTRIBUUTION
NORTHEASTER PT163
SWL = +9.5 FT
MAXIMUM PRESSURES ON CHANNELS 1 AND 2

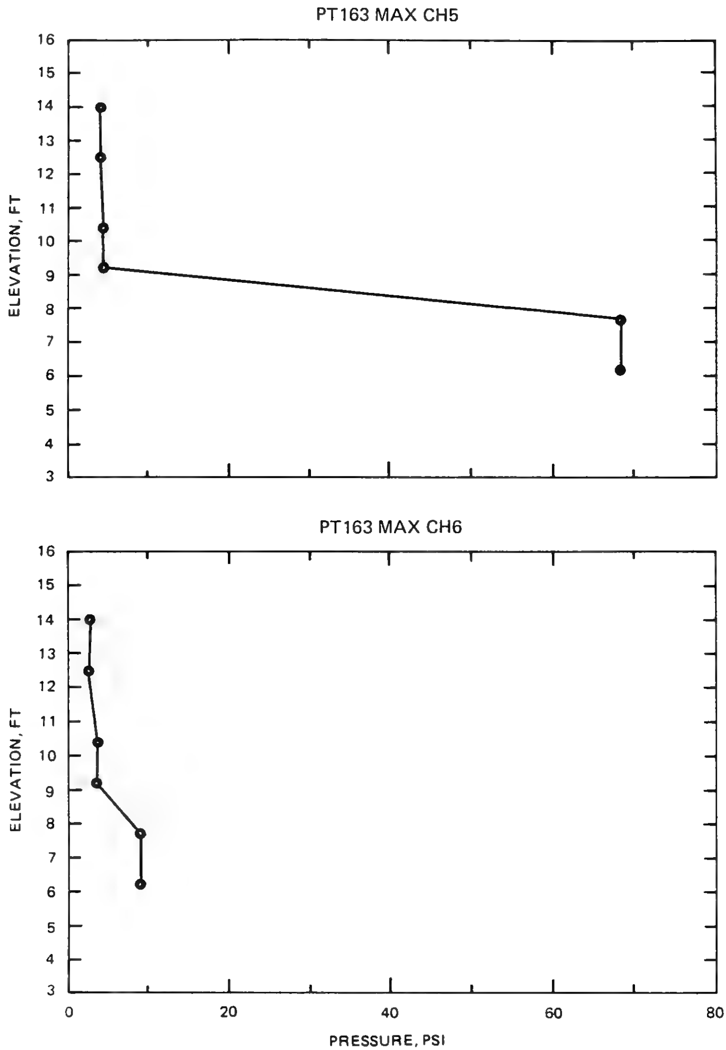


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT163

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4

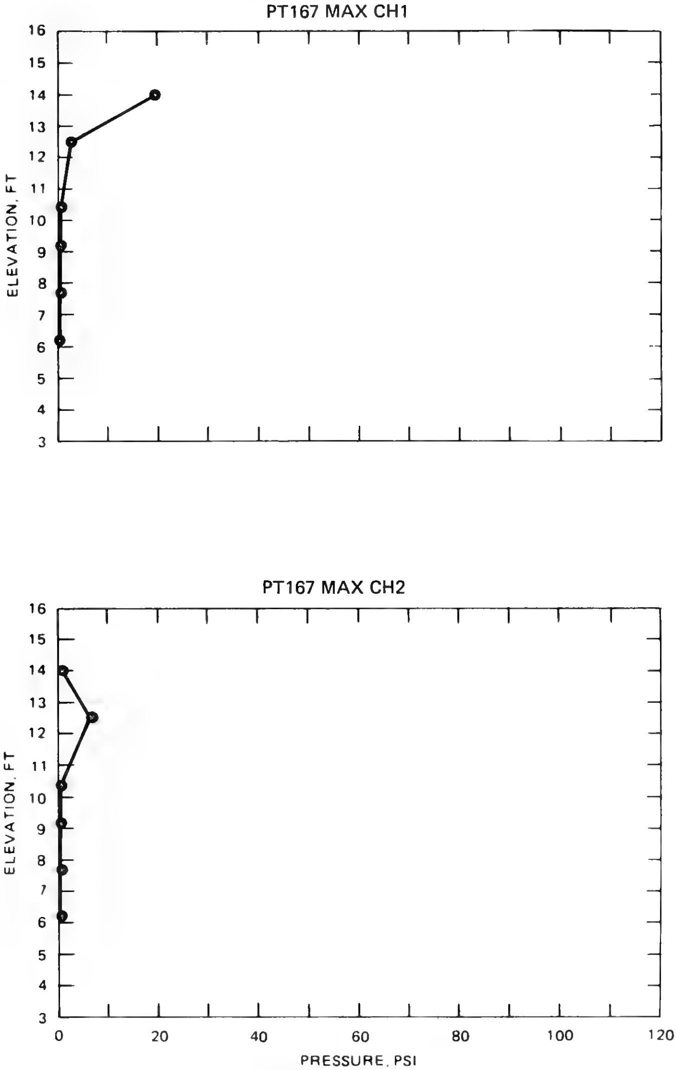


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT163

SWL = +9.5 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

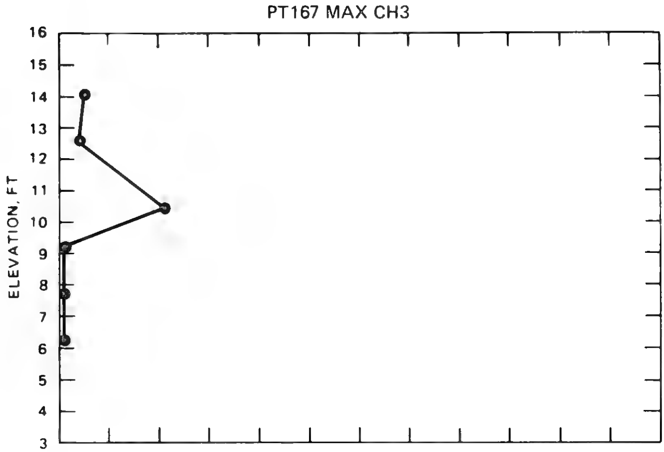


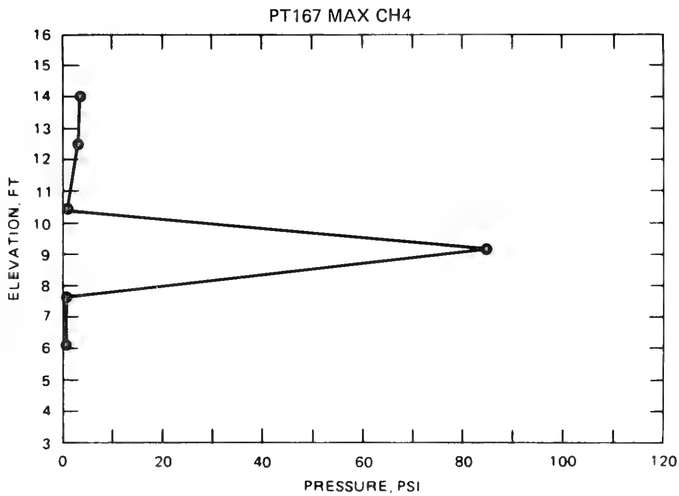
INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT167

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 1 AND 2



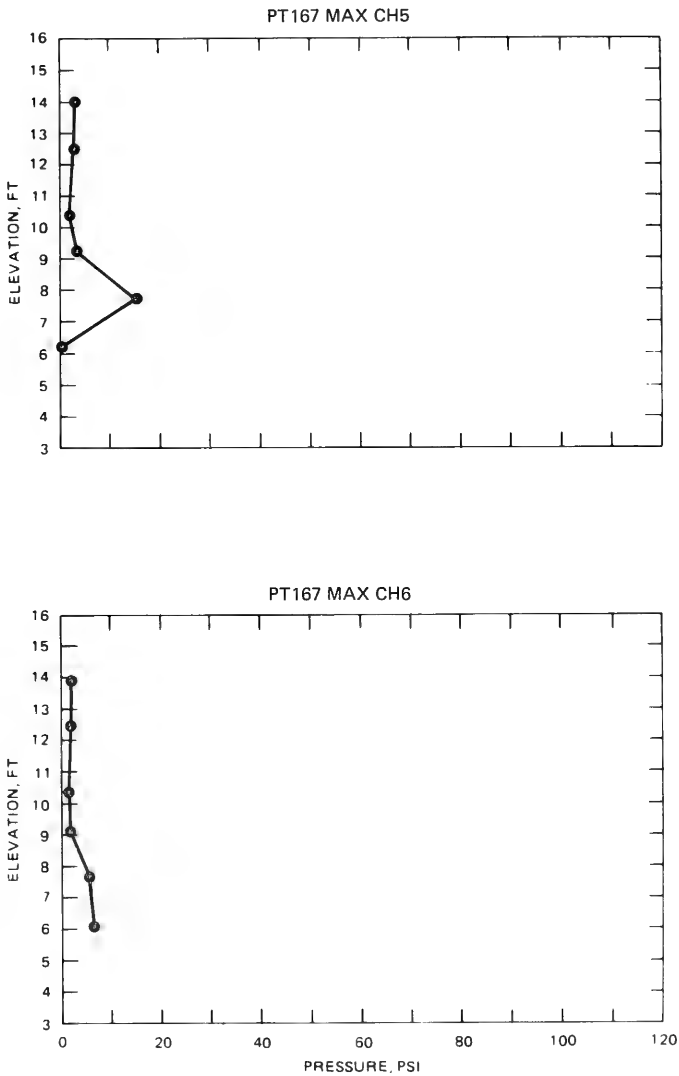


INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT167

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 3 AND 4



INSTANTANEOUS WAVE PRESSURE DISTRIBUTION

NORTHEASTER PT167

SWL = +8.0 FT

MAXIMUM PRESSURES ON CHANNELS 5 AND 6

APPENDIX A: OVERTOPPING TEST RESULTS

Appendix A presents Phase I and Phase II seawall data for two storm types: northeasters (N,NE) and hurricanes (H).

Table A1

Phase I Seawall Data

Test No.	Storm Type	% Gain	SWL with no setup ft.	Setup meas. in tank ft.	Ds effective ft.	Gage Eight Hmo ft.	Gage Eight Tp sec.	Gage Four Hmo ft.	Goda Array2 Hmo ft.	Goda Array2 Tp sec.	Calc. Seiche Hmo ft.	Calc. Seiche Amplitude ft.	Ovtp. rate cfs/ft	Calc. Lp ft.	Rel. Frbd.
75	N	60	9.5	0.1011	8.6	7.51	14.9	4.91	4.35	14.9	2.270	0.803	0.584	248.21	0.3633
76	N	50	9.5	0.0581	8.6	6.29	14.9	4.72	4.24	14.9	2.076	0.734	0.488	247.61	0.3726
77	N	40	9.5	0.0023	8.5	4.97	14.9	4.41	4.02	14.9	1.816	0.642	0.508	246.83	0.3901
60	H	70	9.5	0.2609	8.8	10.13	11.8	5.08	4.41	13.7	2.519	0.891	0.616	230.20	0.3594
65	H	70	9.5	0.2887	8.8	9.94	10.8	4.96	4.31	13.7	2.457	0.869	0.615	230.55	0.3633
67	H	65	9.5	0.0000	8.5	9.25	10.8	4.87	3.62	13.3	3.252	1.150	0.624	220.50	0.4342
66	H	60	9.5	0.2122	8.7	8.64	10.8	4.82	4.29	13.0	2.193	0.775	0.423	218.30	0.3758
40	H	60	9.5	0.2197	8.7	9.77	10.8	5.58	4.86	13.0	2.737	0.968	0.555	218.39	0.3703
68	H	50	9.5	0.1240	8.6	7.26	10.8	4.71	4.21	13.0	2.094	0.740	0.385	217.24	0.3866
37	H	50	9.5	0.1018	8.6	8.19	10.8	5.33	4.80	13.0	2.324	0.822	0.362	216.97	0.3811
39	H	40	9.5	0.0417	8.5	6.55	10.8	5.25	4.69	11.4	2.348	0.830	0.278	188.70	0.4091
61	H	30	9.5	-0.0628	8.4	4.38	10.8	4.26	3.96	13.3	1.578	0.558	0.214	219.72	0.4141
106	N	60	8.0	0.1405	7.1	8.11	14.9	4.71	3.39	15.0	3.273	1.157	0.084	230.49	0.5464
10	N	60	8.0	0.2141	7.2	10.42	15.0	4.67	4.29	15.3	1.838	0.650	0.107	235.73	0.4864
102	N	60	8.0	0.2496	7.2	8.21	14.9	4.73	4.06	17.8	2.426	0.858	0.076	275.66	0.4495
101	N	50	8.0	0.1247	7.1	6.74	15.3	4.44	3.97	15.0	1.994	0.705	0.044	230.25	0.4931
105	N	50	8.0	0.1082	7.1	6.58	15.3	4.41	3.34	15.0	2.871	1.015	0.077	229.99	0.5537
100	N	40	8.0	0.0410	7.0	5.34	15.3	4.13	3.85	15.0	1.497	0.529	0.027	228.96	0.5092
104	N	40	8.0	0.0089	7.0	5.06	15.4	4.06	3.30	15.0	2.368	0.837	0.028	228.46	0.5674
96	N	40	8.0	-0.0558	6.9	5.18	15.3	4.09	3.84	15.0	1.432	0.506	0.025	227.46	0.5183
109	N	40	8.0	0.0482	7.0	5.31	15.3	4.16	3.38	15.0	2.428	0.858	0.030	229.07	0.5548
103	N	30	8.0	-0.0620	6.9	3.84	15.4	3.73	3.58	14.9	1.045	0.369	0.017	224.99	0.5453
110	N	30	8.0	-0.0284	7.0	3.89	15.4	3.84	3.23	14.9	2.082	0.736	0.018	225.50	0.5812
59	H	60	8.0	0.3201	7.3	9.26	12.6	4.57	4.01	22.8	2.200	0.778	0.138	354.92	0.4129
91	H	60	8.0	0.2373	7.2	9.35	11.4	4.63	4.11	11.8	2.137	0.756	0.121	181.75	0.5134
90	H	60	8.0	0.2844	7.3	9.57	11.4	4.70	4.13	10.9	2.229	0.788	0.123	167.74	0.5218
79	H	60	8.0	0.2909	7.3	8.88	11.4	4.40	3.86	22.3	2.105	0.744	0.134	346.90	0.4283
21	H	60	8.0	0.2001	7.2	9.10	10.9	4.52	4.79	14.9	ERR	ERR	0.124	229.77	0.4531
58	H	55	8.0	0.1929	7.2	8.34	12.6	4.41	3.95	11.8	1.961	0.693	0.105	181.23	0.5307
89	H	50	8.0	0.1533	7.2	7.95	11.6	4.43	4.09	11.9	1.699	0.601	0.067	181.77	0.5210
84	H	50	8.0	0.1062	7.1	7.94	12.6	4.42	4.07	15.7	1.710	0.605	0.068	240.98	0.4781
2	H	50	8.0	0.1494	7.1	8.10	11.4	4.67	4.17	15.8	2.102	0.743	0.077	242.57	0.4941
92	H	50	8.0	0.1360	7.1	7.83	11.7	4.40	4.08	11.9	1.650	0.583	0.073	181.57	0.5228
57	H	45	8.0	0.0992	7.1	6.93	11.7	4.29	3.90	11.9	1.781	0.630	0.056	181.13	0.5420
23	H	40	8.0	0.0228	7.0	6.54	12.6	4.61	4.16	13.5	1.986	0.702	0.039	204.80	0.5263
93	H	40	8.0	0.0619	7.1	6.28	11.7	4.31	4.12	13.2	1.265	0.447	0.044	201.54	0.5067
56	H	40	8.0	0.0655	7.1	6.15	11.7	4.26	3.89	12.2	1.740	0.615	0.042	185.93	0.5409
81	H	40	8.0	0.0605	7.1	5.89	11.7	4.04	3.79	13.2	1.394	0.493	0.051	201.52	0.5361
85	H	40	8.0	0.0715	7.1	6.63	11.7	4.37	4.07	12.2	1.605	0.568	0.043	186.00	0.5244
86	H	30	8.0	0.0000	7.0	5.03	12.8	4.24	4.06	12.2	1.225	0.433	0.014	185.11	0.5306
94	H	30	8.0	-0.0205	7.0	4.70	12.8	4.13	3.64	13.5	1.957	0.692	0.014	204.20	0.5536
64	N	60	7.0	0.2534	6.3	8.13	15.2	4.35	3.44	15.0	2.664	0.942	0.025	215.01	0.6187
63	N	50	7.0	0.1552	6.2	6.74	15.2	4.09	3.40	15.0	2.285	0.808	0.010	214.18	0.6318
55	H	60	7.0	0.3595	6.4	9.25	12.8	4.25	3.68	22.2	2.125	0.751	0.039	322.33	0.5101
78	H	60	7.0	0.3588	6.4	9.06	12.9	4.13	3.57	22.2	2.081	0.736	0.036	322.32	0.5210
52	H	40	7.0	0.0643	6.1	6.50	13.1	3.87	3.72	11.9	1.075	0.380	0.007	167.68	0.6518
51	H	30	7.0	-0.0373	6.0	4.79	13.1	3.70	3.61	11.9	0.792	0.280	0.003	165.89	0.6753
12	N	50	6.1	0.1401	5.2	6.85	15.6	3.75	3.12	17.6	2.085	0.737	0.003	231.13	0.7576
13	N	40	6.1	0.0430	5.1	5.56	15.3	3.44	3.02	15.0	1.637	0.579	0.002	194.70	0.8269
33	H	50	6.1	0.1667	5.3	8.32	11.2	4.15	3.45	15.3	2.298	0.813	0.004	201.87	0.7368
32	H	40	6.1	0.0789	5.2	6.91	11.2	3.92	3.54	13.1	1.680	0.594	0.002	171.34	0.7718
34	H	30	6.1	-0.0400	5.0	5.09	13.2	3.67	3.40	13.1	1.379	0.488	0.002	169.45	0.8054

Table A2

Phase II Seawall Data

Test No.	Storm Type	% Gain	SWL with no setup ft.	Setup meas. in tank ft.	Ds effective ft.	Gage Eight Hmo ft.	Gage Eight Tp sec.	Gage Four Hmo ft.	Goda Array2 Hmo ft.	Goda Array2 Tp sec.	Goda Array1 Reflect. Coeff.	Calc. Seiche Hmo ft.	Calc. Seiche Amplitude ft.	Ovtp. rate cfs/ft	Calc. Lp ft.	Rel. Frbd.
195	H	30	7.0	-0.0267	6.0	4.80	11.9	3.91	3.70	10.7	0.5476	1.279	0.452	0.002	153.91	0.6780
196	H	40	7.0	0.0148	6.0	6.24	11.9	4.06	3.79	13.8	0.5474	1.471	0.520	0.008	200.55	0.6082
143	H	40	7.0	0.0289	6.0	6.44	12.0	3.72	3.58	9.9	0.5121	0.992	0.351	0.005	142.95	0.7052
197	H	50	7.0	0.0425	6.0	7.88	11.9	4.11	3.84	13.8	0.5553	1.460	0.516	0.011	200.96	0.6000
144	H	50	7.0	0.1198	6.1	8.08	12.0	3.88	3.63	14.9	0.5305	1.368	0.484	0.008	218.49	0.6001
145	H	60	7.0	0.2385	6.2	9.64	12.0	4.05	3.67	14.9	0.5505	1.712	0.605	0.015	220.37	0.5868
148	H	60	7.0	0.1586	6.2	9.64	11.2	4.52	4.31	14.9	0.5764	1.371	0.485	0.061	218.45	0.5331
198	H	60	7.0	0.2518	6.3	9.56	11.9	4.29	3.95	13.8	0.5632	1.688	0.597	0.019	203.51	0.5727
199	H	70	7.0	0.3431	6.3	11.09	11.9	4.47	3.98	14.6	0.5794	2.047	0.724	0.037	217.60	0.5511
146	H	70	7.0	0.3424	6.3	11.19	12.0	4.27	3.81	16.8	0.5605	1.928	0.682	0.019	250.07	0.5413
200	H	80	7.0	0.4659	6.5	12.64	11.9	4.76	4.10	21.8	0.5982	2.424	0.857	0.083	328.65	0.4637
201	H	90	7.0	0.6264	6.6	14.21	11.9	5.05	4.21	14.5	0.6051	2.780	0.983	0.123	220.63	0.5098
202	H	95	7.0	0.6593	6.7	14.69	11.9	5.11	4.25	16.8	0.6068	2.839	1.004	0.148	255.62	0.4806
134	NE	30	7.0	-0.0654	5.9	3.79	14.9	3.53	3.14	15.6	0.5357	1.619	0.573	0.002	225.91	0.6689
187	NE	30	7.0	-0.0539	5.9	5.14	15.2	3.72	3.40	15.6	0.5531	1.507	0.533	0.004	225.41	0.6339
135	NE	40	7.0	0.0114	6.0	5.14	15.2	3.72	3.40	15.6	0.5392	1.507	0.533	0.004	227.21	0.6275
188	NE	40	7.0	0.0228	6.0	6.55	15.2	3.86	3.55	15.6	0.5449	1.496	0.529	0.011	226.70	0.6084
136	NE	50	7.0	0.0928	6.1	6.55	15.2	3.86	3.55	15.6	0.5433	1.496	0.529	0.004	228.58	0.6018
189	NE	50	7.0	0.0336	6.0	6.47	15.0	3.87	3.52	15.5	0.5466	1.612	0.570	0.001	226.24	0.6120
137	NE	60	7.0	0.1944	6.2	7.93	14.9	4.04	3.60	16.2	0.5456	1.832	0.648	0.008	239.70	0.5800
190	NE	60	7.0	0.1151	6.1	7.78	15.0	4.06	3.58	15.6	0.5693	1.916	0.677	0.009	228.95	0.5974
191	NE	70	7.0	0.3475	6.3	9.20	15.0	4.29	3.71	15.6	0.5857	2.160	0.764	0.023	232.08	0.5650
138	NE	70	7.0	0.2709	6.3	9.21	14.9	4.23	3.68	12.1	0.5622	2.085	0.737	0.019	178.87	0.6245
192	NE	80	7.0	0.4691	6.5	10.62	15.0	4.61	3.89	15.5	0.5867	2.487	0.879	0.048	233.40	0.5385
139	NE	80	7.0	0.3901	6.4	10.60	14.9	4.42	3.85	12.1	0.5779	2.174	0.769	0.045	180.37	0.5964
140	NE	90	7.0	0.5278	6.5	12.07	14.9	4.76	3.98	15.6	0.5887	2.600	0.919	0.077	234.95	0.5246
193	NE	90	7.0	0.5841	6.6	11.87	15.0	4.91	4.02	17.4	0.5897	2.811	0.994	0.094	264.60	0.4978
141	NE	100	7.0	0.6572	6.7	13.38	15.2	5.08	4.11	20.7	0.5982	2.982	1.054	0.137	316.92	0.4574
194	NE	100	7.0	0.0000	6.0	13.22	15.0	5.19	4.16	22.9	0.5981	3.101	1.097	0.149	334.59	0.4829
171	H	30	8.0	-0.0557	6.9	4.69	13.5	4.48	4.15	10.6	0.5986	1.690	0.597	0.039	162.26	0.5513
172	H	40	8.0	-0.0129	7.0	6.32	13.3	4.65	4.48	14.1	0.5878	1.270	0.449	0.063	217.88	0.4728
118	H	40	8.0	0.0930	7.1	6.49	11.2	4.44	4.37	13.9	0.5873	0.787	0.278	0.023	216.95	0.4744
119	H	50	8.0	0.0124	7.0	8.19	11.2	4.56	4.49	14.9	0.5736	0.797	0.282	0.046	230.81	0.4616
173	H	50	8.0	0.0709	7.1	8.12	13.3	4.77	4.55	13.6	0.5887	1.434	0.507	0.075	212.09	0.4666
256	H	60	8.0	0.2818	7.3	9.82	13.3	4.88	4.66	12.5	0.5956	1.431	0.506	0.083	197.14	0.4574
246	H	60	8.0	0.2481	7.2	10.14	13.3	4.92	4.68	12.5	0.5918	1.522	0.538	0.082	196.73	0.4588
251	H	60	8.0	0.1782	7.2	9.80	13.3	4.87	4.60	12.5	0.5974	1.599	0.565	0.093	195.87	0.4690
120	H	60	8.0	0.1246	7.1	9.81	11.2	4.70	4.55	14.9	0.5826	1.171	0.414	0.067	232.48	0.4495
261	H	60	8.0	0.1211	7.1	9.64	13.3	4.96	4.76	12.5	0.5985	1.385	0.490	0.125	195.16	0.4628
174	H	60	8.0	0.1856	7.2	9.81	13.3	4.95	4.60	12.5	0.594	1.837	0.649	0.113	195.96	0.4687
252	H	70	8.0	0.2791	7.3	11.44	13.3	5.10	4.83	15.6	0.6004	1.631	0.577	0.151	246.08	0.4152
149	H	70	8.0	0.3065	7.3	11.32	11.2	4.72	4.42	14.8	0.5786	1.667	0.589	0.095	233.83	0.4466
121	H	70	8.0	0.3304	7.3	11.23	11.2	4.91	4.70	14.9	0.5911	1.411	0.499	0.122	235.51	0.4151
257	H	70	8.0	0.3770	7.4	11.38	13.3	5.11	4.84	14.5	0.6011	1.639	0.579	0.136	230.85	0.4179
262	H	70	8.0	0.2415	7.2	11.14	13.3	5.17	4.90	15.6	0.6089	1.636	0.579	0.209	245.50	0.4137
247	H	70	8.0	0.2816	7.3	11.66	13.3	5.07	4.83	14.5	0.5937	1.527	0.540	0.121	229.48	0.4249
175	H	70	8.0	0.2970	7.3	11.42	13.3	5.19	4.77	15.7	0.603	2.058	0.728	0.186	247.89	0.4170
150	H	80	8.0	0.4157	7.4	12.59	11.2	5.19	4.73	14.9	0.5956	2.135	0.755	0.185	236.76	0.4188
258	H	80	8.0	0.4790	7.5	12.89	13.3	5.38	4.92	15.6	0.6044	2.181	0.771	0.230	249.14	0.3975
176	H	80	8.0	0.4131	7.4	12.97	13.3	5.48	4.87	14.5	0.5963	2.512	0.888	0.306	231.36	0.4141

(Continued)

(Sheet 1 of 3)

Table A2 (Continued)

Test No.	Storm Type	% Gain	SWL with no setup ft.	Setup meas. in tank ft.	Ds effective ft.	Gage Eight Hmo ft.	Gage Eight Tp sec.	Gage Four Hmo ft.	Goda Array2 Hmo ft.	Goda Array2 Tp sec.	Goda Array1 Reflect. Coeff.	Calc. Seiche Hmo ft.	Calc. Seiche Amplitude ft.	Ovtp. rate cfs/ft	Calc. Lp ft.	Rel. Frbd.
122	H	80	8.0	0.0000	7.0	13.06	11.2	5.18	4.87	14.5	0.6059	1.770	0.626	0.182	225.20	0.4303
248	H	80	8.0	0.3863	7.4	13.28	13.3	5.36	4.92	14.5	0.6011	2.124	0.751	0.231	230.98	0.4128
253	H	80	8.0	0.3844	7.4	12.99	13.3	5.39	4.85	15.6	0.5999	2.334	0.825	0.245	247.70	0.4071
263	H	80	8.0	0.3399	7.3	12.59	13.3	5.44	4.97	14.5	0.6084	2.197	0.777	0.315	230.32	0.4129
254	H	90	8.0	0.4862	7.5	14.43	13.3	5.63	4.92	11.5	0.6089	2.732	0.966	0.405	182.64	0.4401
123	H	90	8.0	0.6230	7.6	14.18	11.2	5.40	4.92	10.4	0.6078	2.228	0.788	0.308	166.02	0.4339
177	H	90	8.0	0.5480	7.5	14.64	13.3	5.73	4.97	21.8	0.6114	2.841	1.004	0.430	351.52	0.3485
151	H	90	8.0	0.5284	7.5	14.01	11.2	5.47	4.81	10.1	0.5983	2.614	0.924	0.273	161.16	0.4636
249	H	90	8.0	0.4799	7.5	14.69	13.3	5.61	4.94	10.9	0.6096	2.664	0.942	0.308	173.17	0.4476
259	H	90	8.0	0.6280	7.6	14.42	13.3	5.69	4.97	11.2	0.6057	2.767	0.978	0.355	179.31	0.4314
264	H	90	8.0	0.4585	7.5	13.99	13.3	5.68	5.05	10.9	0.6159	2.609	0.922	0.479	172.95	0.4426
153	H	95	8.0	0.5806	7.6	14.74	11.2	5.53	4.85	9.9	0.6064	2.650	0.937	0.307	157.85	0.4605
125	H	95	8.0	0.7203	7.7	14.92	11.2	5.57	5.04	10.1	0.6102	2.367	0.837	0.346	163.06	0.4236
178	H	95	8.0	0.0000	7.0	15.45	13.3	5.87	5.03	21.8	0.6136	3.024	1.069	0.535	339.58	0.3769
260	H	100	8.0	0.7292	7.7	15.61	10.9	5.90	5.06	10.9	0.6236	3.041	1.075	0.480	175.75	0.4231
265	H	100	8.0	0.5730	7.6	15.17	13.3	5.92	5.04	10.9	0.6149	3.095	1.094	0.585	174.14	0.4349
255	H	100	8.0	0.5744	7.6	15.73	13.3	5.88	4.98	11.5	0.6216	3.131	1.107	0.197	183.60	0.4308
250	H	100	8.0	0.5624	7.6	15.83	13.3	5.81	4.98	10.9	0.6147	2.994	1.058	0.442	174.03	0.4391
179	NE	30	8.0	-0.1085	6.9	3.87	14.9	4.26	3.87	15.4	0.6179	1.783	0.630	0.047	237.17	0.5005
126	NE	30	8.0	-0.0454	7.0	3.89	15.1	4.24	3.77	15.4	0.6015	1.944	0.687	0.012	238.15	0.5165
180	NE	40	8.0	-0.1041	6.9	5.15	14.9	4.66	4.24	15.5	0.6019	1.928	0.682	0.095	238.67	0.4697
181	NE	50	8.0	-0.0617	6.9	6.60	14.7	4.80	4.25	15.6	0.5876	2.236	0.791	0.122	240.77	0.4650
128	NE	50	8.0	-0.0377	7.0	6.83	14.7	4.74	4.36	15.6	0.5983	1.873	0.662	0.050	241.15	0.4669
129	NE	60	8.0	0.1009	7.1	7.99	14.7	4.84	4.54	15.6	0.587	1.667	0.589	0.123	243.32	0.4448
182	NE	60	8.0	0.1876	7.2	8.02	14.7	4.84	4.53	15.6	0.5832	1.724	0.610	0.163	244.67	0.4287
183	NE	70	8.0	0.2730	7.3	9.44	14.9	5.00	4.55	15.5	0.592	2.078	0.735	0.208	245.29	0.4218
130	NE	70	8.0	0.1886	7.2	9.59	14.9	4.99	4.62	15.5	0.5889	1.882	0.666	0.169	243.99	0.4340
131	NE	80	8.0	0.3122	7.3	10.96	14.9	5.28	4.70	16.2	0.6004	2.404	0.850	0.230	257.51	0.4148
184	NE	80	8.0	0.3511	7.4	10.81	14.9	5.26	4.61	16.2	0.598	2.528	0.894	0.279	258.13	0.4069
185	NE	90	8.0	0.4298	7.4	12.04	14.9	5.49	4.71	16.2	0.6124	2.814	0.995	0.387	259.39	0.3960
132	NE	90	8.0	0.4585	7.5	12.41	14.9	5.56	4.87	16.2	0.6149	2.690	0.951	0.324	259.85	0.3958
186	NE	100	8.0	0.5518	7.6	13.76	14.9	5.90	5.05	25.8	0.6171	3.050	1.078	0.513	416.09	0.3172
133	NE	100	8.0	0.5696	7.6	13.76	14.9	5.90	5.05	17.4	0.6304	3.050	1.078	0.448	281.01	0.3705
162	H	30	9.5	-0.0187	8.5	4.84	13.3	4.99	4.60	14.6	0.6341	1.923	0.680	0.137	247.06	0.3579
163	H	40	9.5	-0.0551	8.4	6.57	13.3	5.29	5.10	14.7	0.6052	1.405	0.497	0.278	247.92	0.3356
164	H	50	9.5	0.0099	8.5	8.32	14.7	5.33	5.18	14.7	0.5979	1.224	0.433	0.332	248.79	0.3283
241	H	60	9.5	0.0944	8.6	10.46	13.3	5.75	5.52	14.7	0.5931	1.603	0.567	0.537	249.92	0.3100
203	H	60	9.5	0.0960	8.6	9.51	13.3	5.48	5.30	14.7	0.5908	1.387	0.491	0.447	249.94	0.3185
221	H	60	9.5	0.0077	8.5	10.21	10.9	5.77	5.57	14.7	0.5947	1.525	0.539	0.834	248.76	0.3132
165	H	60	9.5	0.0787	8.6	9.90	13.3	5.49	5.25	14.7	0.5929	1.629	0.576	0.413	249.71	0.3217
222	H	70	9.5	0.1037	8.6	11.96	13.3	5.97	5.65	14.3	0.5984	1.903	0.673	0.582	244.52	0.3069
204	H	70	9.5	0.1541	8.7	11.07	13.3	5.63	5.30	15.6	0.5984	1.896	0.670	0.547	266.66	0.3086
232	H	70	9.5	0.1562	8.7	11.84	10.9	5.85	5.54	14.7	0.5944	1.867	0.660	0.666	250.74	0.3057
166	H	70	9.5	0.2029	8.7	11.49	10.9	5.67	5.32	14.7	0.6014	1.960	0.693	0.537	251.36	0.3115
242	H	70	9.5	0.1674	8.7	12.04	10.9	5.86	5.57	14.3	0.5925	1.835	0.649	0.685	245.34	0.3064
243	H	80	9.5	0.2626	8.8	13.57	13.3	6.07	5.57	15.6	0.5996	2.403	0.850	0.780	268.19	0.2926
205	H	80	9.5	0.2396	8.7	12.65	13.3	5.79	5.37	15.5	0.6058	2.184	0.772	0.648	266.26	0.3020
233	H	80	9.5	0.2499	8.7	13.31	13.3	6.00	5.59	15.6	0.5975	2.170	0.767	0.694	268.01	0.2927
223	H	80	9.5	0.2103	8.7	13.53	13.3	6.16	5.68	15.6	0.5962	2.390	0.845	1.029	267.45	0.2919
167	H	80	9.5	0.3175	8.8	13.09	13.3	5.86	5.38	15.6	0.6065	2.333	0.825	0.634	268.96	0.2967

(Continued)

(Sheet 2 of 3)

Test No.	Storm Type	% Gain	SWL with no setup ft.	Setup meas. in tank ft.	Ds effective ft.	Gage Eight Hmo ft.	Gage Eight Tp sec.	Gage Four Hmo ft.	Goda Array2 Hmo ft.	Goda Array2 Tp sec.	Goda Array1 Reflect. Coeff.	Calc. Seiche Hmo ft.	Calc. Seiche Amplitude ft.	Ovtp. rate cfs/ft	Calc. Lp ft.	Rel. Frbd.
244	H	90	9.5	0.3515	8.9	15.03	13.3	6.32	5.64	14.5	0.6115	2.860	1.011	0.856	251.17	0.2924
234	H	90	9.5	0.3798	8.9	14.92	10.9	6.30	5.67	14.5	0.6143	2.740	0.969	0.944	251.54	0.2896
224	H	90	9.5	0.2928	8.8	14.93	13.3	6.43	5.79	15.3	0.6087	2.802	0.991	1.249	264.63	0.2850
206	H	90	9.5	0.2899	8.8	14.65	13.3	6.06	5.49	15.3	0.6116	2.565	0.907	0.760	264.60	0.2956
168	H	90	9.5	0.4197	8.9	14.57	13.3	6.13	5.47	15.3	0.6093	2.753	0.973	0.793	266.38	0.2890
170	H	95	9.5	0.4437	8.9	15.29	13.3	6.26	5.55	15.6	0.6096	2.886	1.020	0.787	270.72	0.2834
235	H	100	9.5	0.4526	9.0	16.34	13.3	6.44	5.75	18.2	0.6126	2.905	1.027	1.058	316.67	0.2623
209	H	100	9.5	0.2398	8.7	15.90	13.3	6.09	5.46	15.0	0.6172	2.699	0.954	1.040	258.49	0.3015
245	H	100	9.5	0.4535	9.0	16.34	10.9	6.50	5.75	15.6	0.613	3.032	1.072	0.704	270.86	0.2764
225	H	100	9.5	0.3545	8.9	16.09	13.3	6.52	5.81	14.5	0.6107	2.944	1.041	1.157	251.21	0.2863
154	NE	30	9.5	-0.1413	8.4	3.83	15.7	4.48	4.39	14.8	0.6299	0.871	0.308	0.262	248.93	0.3754
155	NE	40	9.5	-0.1202	8.4	5.54	15.5	5.00	4.90	15.5	0.6053	0.989	0.350	0.228	262.00	0.3420
156	NE	50	9.5	-0.1018	8.4	6.85	14.4	5.20	5.06	15.5	0.5925	1.209	0.428	0.425	262.26	0.3337
211	NE	60	9.5	0.0059	8.5	8.04	14.4	5.36	5.10	15.5	0.5874	1.668	0.590	0.406	262.97	0.3262
236	NE	60	9.5	0.0396	8.5	7.97	15.5	5.57	5.21	15.5	0.5866	1.988	0.703	0.546	263.45	0.3195
157	NE	60	9.5	-0.0620	8.4	8.36	14.4	5.39	5.21	15.5	0.5846	1.354	0.479	0.533	262.01	0.3252
216	NE	60	9.5	0.0086	8.5	8.19	14.4	5.49	5.14	15.5	0.5903	1.922	0.680	0.473	263.01	0.3242
226	NE	60	9.5	-0.1511	8.3	8.06	14.4	5.68	5.26	15.5	0.5876	2.157	0.763	0.658	261.55	0.3282
212	NE	70	9.5	0.0589	8.6	9.34	15.1	5.48	5.12	15.5	0.5854	1.946	0.688	0.493	263.72	0.3219
237	NE	70	9.5	0.1210	8.6	9.45	15.5	5.67	5.25	15.5	0.5851	2.156	0.762	0.590	264.60	0.3133
227	NE	70	9.5	-0.0758	8.4	9.43	15.5	5.75	5.33	15.5	0.5827	2.171	0.768	0.939	261.81	0.3214
217	NE	70	9.5	0.1066	8.6	9.61	15.5	5.63	5.23	15.5	0.592	2.101	0.743	0.565	264.39	0.3149
158	NE	70	9.5	0.0060	8.5	9.82	15.5	5.49	5.24	15.5	0.5851	1.647	0.582	0.567	262.97	0.3201
228	NE	80	9.5	0.0011	8.5	10.89	15.5	5.96	5.41	15.5	0.5769	2.508	0.887	0.883	262.90	0.3136
218	NE	80	9.5	0.1703	8.7	11.01	15.5	5.79	5.28	15.5	0.5849	2.369	0.838	0.651	265.29	0.3092
159	NE	80	9.5	0.0931	8.6	11.26	15.5	5.67	5.33	15.5	0.5833	1.924	0.680	0.653	264.20	0.3115
238	NE	80	9.5	0.2284	8.7	10.94	15.5	5.84	5.36	15.5	0.591	2.324	0.822	0.573	266.10	0.3030
239	NE	90	9.5	0.3213	8.8	12.37	15.1	6.08	5.48	17.4	0.5895	2.627	0.929	0.832	301.83	0.2815
219	NE	90	9.5	0.1831	8.7	12.07	15.1	6.21	5.53	14.5	0.5856	2.838	1.003	1.105	248.97	0.3056
214	NE	90	9.5	0.3743	8.9	12.48	15.1	5.95	5.43	19.0	0.5926	2.438	0.862	0.675	329.33	0.2729
160	NE	90	9.5	0.2683	8.8	12.56	15.1	5.90	5.44	15.5	0.5949	2.288	0.809	0.612	267.50	0.2974
229	NE	90	9.5	0.3046	8.8	12.63	15.1	6.12	5.45	19.0	0.5886	2.774	0.981	0.826	328.13	0.2756
230	NE	100	9.5	0.3437	8.8	13.82	15.1	6.31	5.56	19.0	0.5915	3.000	1.061	0.790	328.80	0.2702
161	NE	100	9.5	0.3540	8.9	13.94	15.1	6.16	5.57	13.2	0.5971	2.629	0.929	0.941	227.92	0.3042
220	NE	100	9.5	0.2550	8.8	13.42	15.1	6.42	5.60	18.9	0.5886	3.152	1.114	1.127	327.20	0.2734
215	NE	100	9.5	0.4676	9.0	13.79	15.1	6.22	5.51	19.0	0.5971	2.890	1.022	0.833	330.92	0.2655
240	NE	100	9.5	0.3914	8.9	13.55	15.1	6.35	5.55	19.0	0.5949	3.074	1.087	0.966	329.62	0.2679

(Sheet 3 of 3)

APPENDIX B: COMPARISON OF SEAWALL PERFORMANCE AND BEACH EROSION EFFECTS

1. To compare the relative effectiveness of the two seawall geometries and to estimate the overtopping rates for a beach erosion level different from the one that was tested, regression coefficients must be determined for Equation 3 (main text) for each data set. Table B1 lists the regression coefficients determined for the Phase I seawall data set (A), the Phase II seawall data set (B), a partial Phase II data set (C) containing only the test results for the 100 percent values of the design wave height at the wave board (DWHAWB), and a partial Phase II data set (D) containing test results for only those tests with a 70 percent or less DWHAWB.

Table B1

Regression Coefficients for Phase I and Phase II Data Sets

Data Set	Regression Coefficient: Q_o, cfs/ft	Regression Coefficient: C_1
A, Phase I Seawall 30 to 70% DWHAWB	52.3	-13.19
B, Phase II Seawall 30 to 100% DWHAWB	50.4	-13.67
C, Phase II Seawall 100% DWHAWB	7.78	-7.476
D, Phase II Seawall 30 to 70% DWHAWB	37.94	-13.89

The data curves for data sets A, B, C, and D have been drawn through the data and are shown in Figures B1, B2, B3, and B4, respectively.

2. Comparisons of the overtopping rates between the two seawall geometries and the different beach erosion levels can be made by computing a maximum zero-moment wave height $H_{mo_{max}}$ at the structure toe. From earlier wave tank tests (Ahrens and Heimbaugh, in publication*) it has been found that the

* References cited in the Appendixes can be found in the References at the end of the main text.

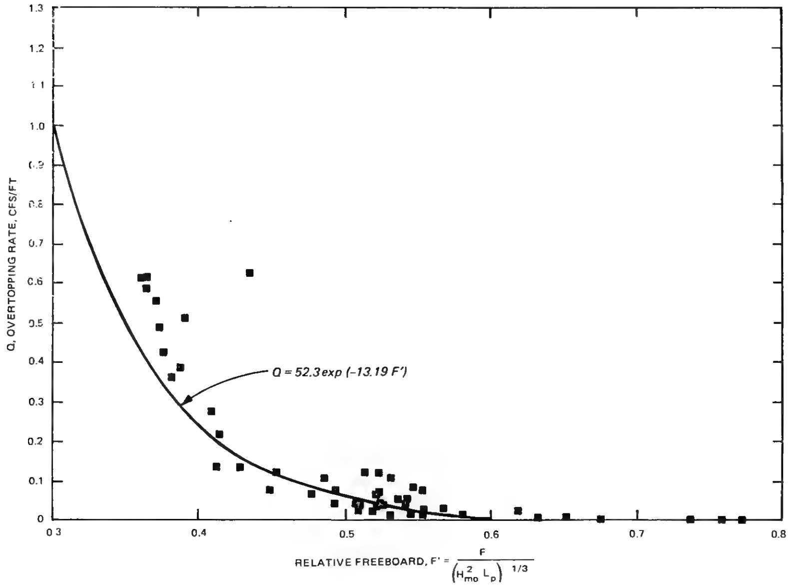


Figure B1. Q versus F' data plot for Phase I seawall, data set A (30 to 70 percent DWHAWB)

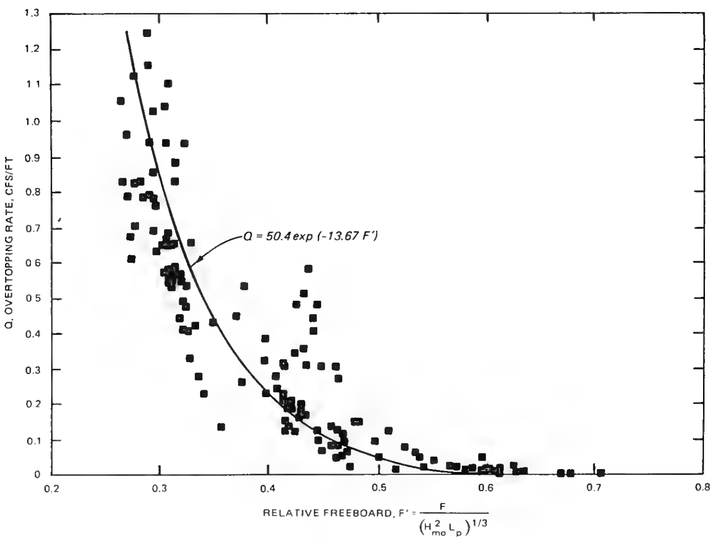


Figure B2. Q versus F' for Phase II seawall, data set B (30 to 100 percent DWHAWB)

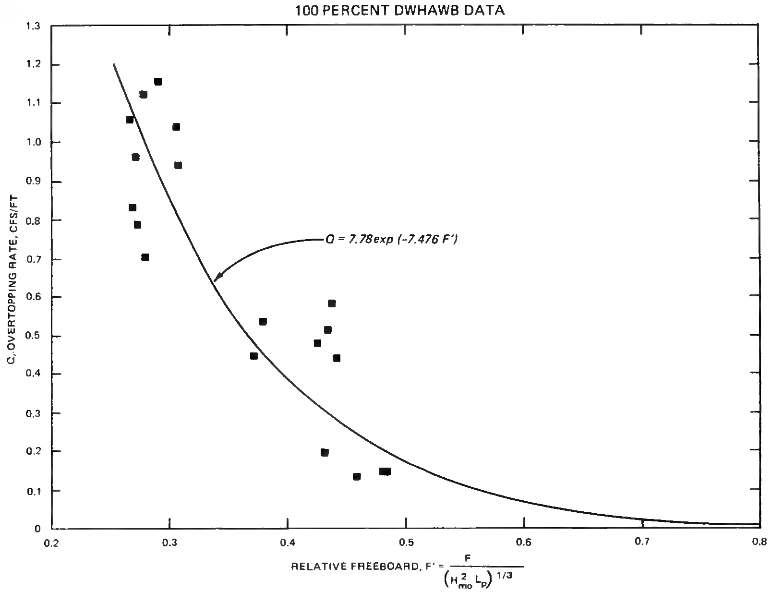


Figure B3. Q versus F' data plot for Phase II seawall, data set C (100 percent DWHAWB)

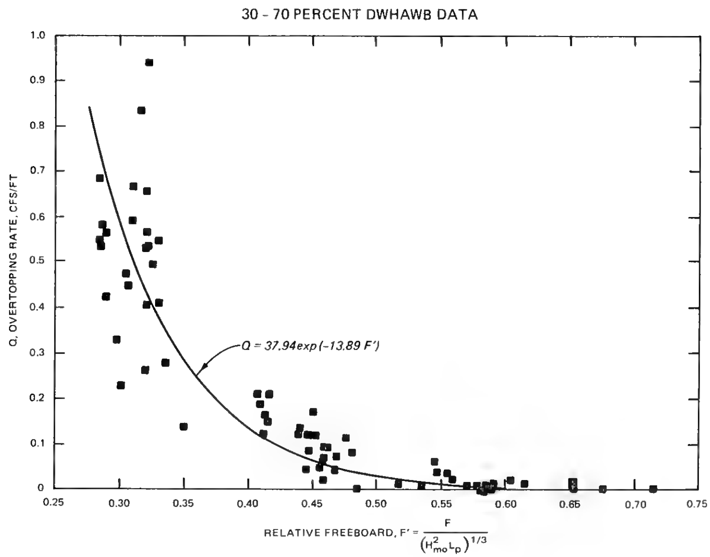


Figure B4. Q versus F' for Phase II seawall, data set D (30 to 70 percent DWHAWB)

approximate limiting value for the zero-moment wave height is given by

$$\left(\frac{H_{mo}}{L_p}\right)_{max} = 0.10 \tan h \left(\frac{2\pi d_s}{L_p}\right) \tag{B1}$$

By substituting this $(H_{mo})_{max}$ value into Equation 1 (main text), a corresponding relative freeboard parameter F' can be determined. This value of F' then can be substituted into Equation 3 (main text), and a Q value can be calculated or read from the appropriate data curve (Figures B1, B2, B3, and B4). This Q value is representative of an average overtopping rate associated with a maximum H_{mo} for a specific local still-water level (swl) at the structure toe d_s. The d_s value used in these calculations should include an estimate of the setup which could occur at the project site. The Q values determined in this manner can then be intercompared, and percent differences and/or percent decreases in the overtopping rates can be computed.

3. Overtopping rates calculated using data sets A, B, and D are listed in Table B2. Phase I and Phase II seawall comparisons expressed in percent decrease in Q for the hurricane event at the three swl's tested are given in Table 2 of the main text.

4. Overtopping rates calculated using data sets B and C are listed in Table B3. The percent difference and percent decrease in Q given in Table B4 and Table 3 (main text) were determined using the calculated Q values for the hurricane conditions in Table B3.

Table B2

Calculated Q Values for Comparison of Phase I and Phase II Seawalls

Data Set	Storm Type*	swl with No Setup ft	swl D_s ft	Setup (Measured in Tank) ft	D_s Effective ft	T_p sec	L_p ft	F Average Freeboard ft	F' Relative Freeboard	H_{mo} max ft	Q Predicted cfs/ft
				Phase One Seawall (30 to 70% Gains)							
A	H	9.5	8.5	0.4	8.9	13.7	224.36	6.2	0.338	5.242	0.605
	NE	9.5	8.5	0.4	8.9	15.4	252.70	6.2	0.324	5.263	0.727
	H	8.0	7.0	0.6	7.6	13.7	203.94	7.7	0.492	4.331	0.079
	NE	8.0	7.0	0.6	7.6	15.4	229.62	7.7	0.472	4.345	0.103
	H	7.0	6.0	0.6	6.6	13.7	189.02	8.7	0.631	3.721	0.013
	NE	7.0	6.0	0.6	6.6	15.4	212.78	8.7	0.606	3.731	0.018
				Phase Two Seawall (30 to 100% Gains)							
B	H	9.5	8.5	0.4	8.9	13.7	224.36	6.2	0.338	5.242	0.496
	NE	9.5	8.5	0.4	8.9	15.4	252.70	6.2	0.324	5.263	0.600
	H	8.0	7.0	0.6	7.6	13.7	203.94	7.7	0.492	4.331	0.060
	NE	8.0	7.0	0.6	7.6	15.4	229.62	7.7	0.472	4.345	0.079
	H	7.0	6.0	0.6	6.6	13.7	180.02	8.7	0.631	3.721	0.009
	NE	7.0	6.0	0.6	6.6	15.4	212.78	8.7	0.606	3.731	0.013
				Phase Two Seawall (30 to 70% Gains Only)							
D	H	9.5	8.5	0.4	8.9	13.7	224.36	6.2	0.338	5.242	0.346
	NE	9.5	8.5	0.4	8.9	15.4	252.70	6.2	0.324	5.263	0.421
	H	8.0	7.0	0.6	7.6	13.7	203.94	7.7	0.492	4.331	0.041
	NE	8.0	7.0	0.6	7.6	15.4	229.62	7.7	0.472	4.345	0.054
	H	7.0	6.0	0.6	6.6	13.7	180.02	8.7	0.631	3.721	0.006
	NE	7.0	6.0	0.6	6.6	15.4	212.78	8.7	0.606	3.731	0.008

* H = hurricane; NE = northeaster.

Table B3

Calculated Q Values for Effect of Beach Elevation* on Overtopping Rates

Data Set	Storm Type**	swl with No Setup ft	swl D_s ft	Setup (Measured in Tank) ft	D_s Effective ft	T_p sec	L_p ft	F Average Freeboard ft	F' Relative Freeboard	H_{mo} max ft	Q Predicted cfs/ft
				Phase Two Seawall (All Gains +1.0 Beach Elevation)							
B	H	9.5	8.5	0.4	8.9	13.7	224.36	6.2	0.338	5.242	0.496
	NE	9.5	8.5	0.4	8.9	15.4	252.70	6.2	0.324	5.263	0.600
	H	8.0	7.0	0.6	7.6	13.7	203.94	7.7	0.492	4.331	0.060
	NE	8.0	7.0	0.6	7.6	15.4	229.62	7.7	0.472	4.345	0.079
	H	7.0	6.0	0.6	6.6	13.7	189.02	8.7	0.631	3.721	0.009
	NE	7.0	6.0	0.6	6.6	15.4	212.78	8.7	0.606	3.731	0.013
				Phase Two Seawall (All Gains +3.4 Beach Elevation)							
	H	9.5	6.1	0.4	6.5	13.7	190.57	6.2	0.444	3.782	0.117
	NE	9.5	6.1	0.4	6.5	15.4	214.52	6.2	0.426	3.792	0.149
	H	8.0	4.6	0.6	5.2	13.7	165.77	7.7	0.695	2.861	0.004
	NE	8.0	4.6	0.6	5.2	15.4	186.54	7.7	0.668	2.867	0.005
	H	7.0	3.6	0.6	4.2	13.6	146.81	8.7	0.962	2.244	0.000
	NE	7.0	3.6	0.6	4.2	15.4	165.17	8.7	0.924	2.248	0.000
				Phase Two Seawall (100% Gains +1.0 Beach Elevation)							
C	H	9.5	8.5	0.4	8.9	13.7	224.36	6.2	0.338	5.242	0.621
	NE	9.5	8.5	0.4	8.9	15.4	252.70	6.2	0.324	5.263	0.690
	H	8.0	7.0	0.6	7.6	13.7	203.94	7.7	0.492	4.331	0.196
	NE	8.0	7.0	0.6	7.6	15.4	229.62	7.7	0.472	4.345	0.228
	H	7.0	6.0	0.6	6.6	13.7	180.02	8.7	0.631	3.721	0.069
	NE	7.0	6.0	0.6	6.6	15.4	212.78	8.7	0.606	3.731	0.084
				Phase Two Seawall (100% Gains +3.4 Beach Elevation)							
	H	9.5	6.1	0.4	6.5	13.7	190.57	6.2	0.444	3.782	0.282
	NE	9.5	6.1	0.4	6.5	15.4	214.52	6.2	0.426	3.792	0.322
	H	8.0	4.6	0.6	5.2	13.7	165.77	7.7	0.695	2.861	0.043
	NE	8.0	4.6	0.6	5.2	15.4	186.54	7.7	0.668	2.867	0.053
	H	7.0	3.6	0.6	4.2	13.7	146.81	8.7	0.962	2.244	0.006
	NE	7.0	3.6	0.6	4.2	15.4	165.17	8.7	0.924	2.248	0.008

* All elevations (el) cited herein are in feet referenced to National Geodetic Vertical Datum (NGVD).
** H = hurricane; NE = northeaster.

Table B4

Overtopping Comparisons Using Hurricane Conditions and the +1.0- and +3.4-ft NGVD Beach Elevation Levels

	Percent Difference*		Percent Decrease**	
swl ft	Phase II 100% Data† %	Phase II 30 to 100% Data %††	Phase II 100% Data %	Phase II 30 to 100% Data %
+9.5	46	24	54	76
+8.0	22	6	78	94
+7.0	9	0	91	100

* Percent difference in Q for data at +1.0-ft NGVD beach elevation and predicted data at +3.4-ft NGVD beach elevation.

** Percent decrease in Q for +3.4-ft NGVD beach elevation versus +1.0-ft NGVD beach elevation.

† Percents are based on Q values calculated in Table B3 using only the 100 percent DWHAWB data points (100% data).

†† Percents are based on Q values calculated in Table B3 using all the data points (30 to 100% data).

APPENDIX C: WAVE SETUP AND SEICHE EFFECTS

1. During analysis of the overtopping test results, it was determined that wave setup and seiche were present in the wave tank. Wave setup is the superelevation of the water surface above normal still-water levels (swl's) and is related to wave breaking. Wave setup is caused by the radiation stress (wave-induced transport or momentum) of the waves progressing toward the shore (Seelig and Ahrens 1980). A seiche is a long-period standing wave which occurs in an enclosed body of water such as a wave tank. Seiches are commonly found in the prototype in lakes and embayments; consequently, they are unlikely to be found along the open coast of Virginia Beach.

Wave Setup

2. Wave setup which occurred in the wave tank increased as the percent gain of the design wave height at the wave board (DWHAWB) increased. Setup became significant only at the higher gain settings. Wave setups as high as 0.3 and 0.7 ft (prototype) were reached in the Phase I and Phase II studies, respectively (Figures C1 and C2). As would be expected, the setup was greater for the lower swl conditions of +8.0 and +7.0 ft National Geodetic Vertical Datum (NGVD). The effect that setup had in the model was to effectively increase the local swl at the structure. This increase was accounted for in the relative freeboard parameter F' by simply adding the measured setup to d_s, the water depth at the structure toe. By increasing d_s, the average freeboard F and significant wavelength L_p were also adjusted. Accounting for setup in the data analysis in this manner implies that wave setup which occurred in the wave tank was the best estimate of wave setup that would occur at Virginia Beach.

Wave Tank Seiche

3. Initial data analysis showed that a seiche was occurring in the wave tank, and it was determined that much of the data scatter seen in Figures 8 and 9 (main text) was due to this seiche. Figures C3 and C4 show the calculated seiche wave amplitude plotted versus the percent gain DWHAWB. The figures clearly show how the seiche wave amplitude increased as the percent gain

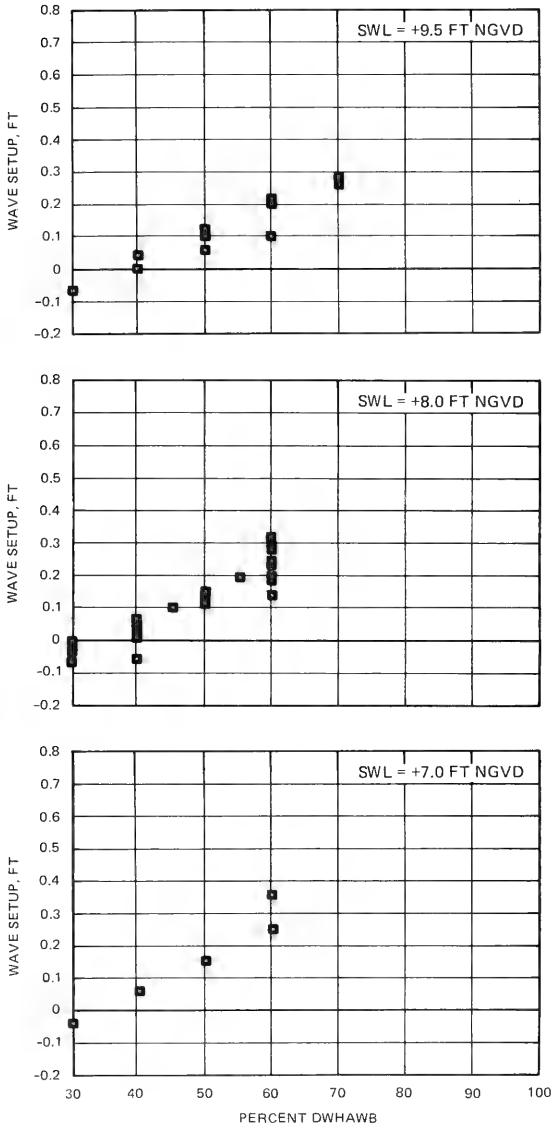


Figure C1. Setup versus percent gain (DWHAWB) at swl's of +9.5, +8.0, and +7.0 ft NGVD, Phase I

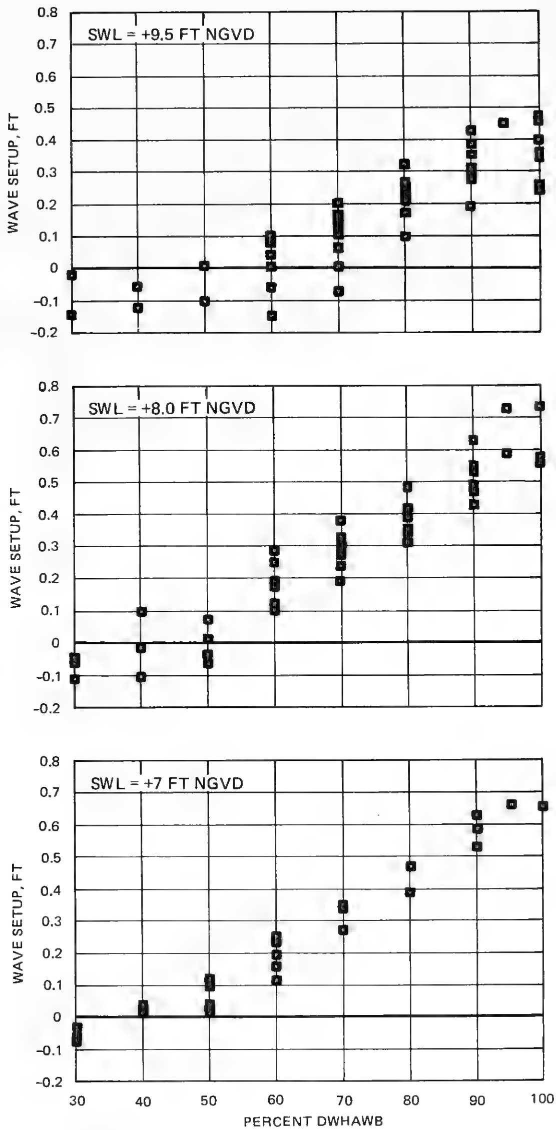


Figure C2. Setup versus percent gain (DWHAWB) at swl's of +9.5, +8.0, +7.0, and +6.0 ft NGVD, Phase II seawall

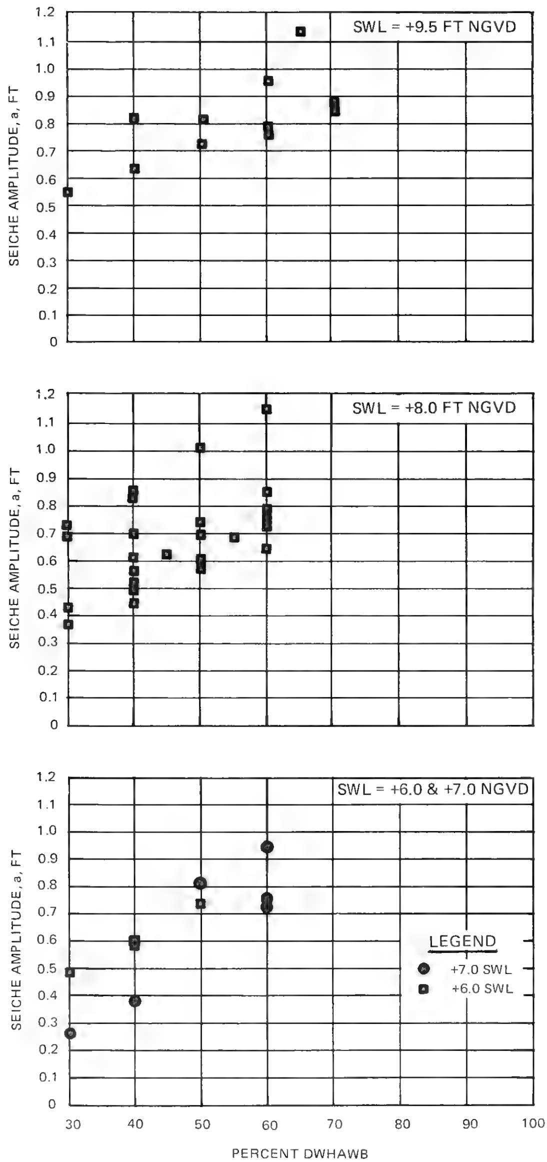


Figure C3. Seiche amplitude versus percent gain (DWHAWB) for Phase I seawall

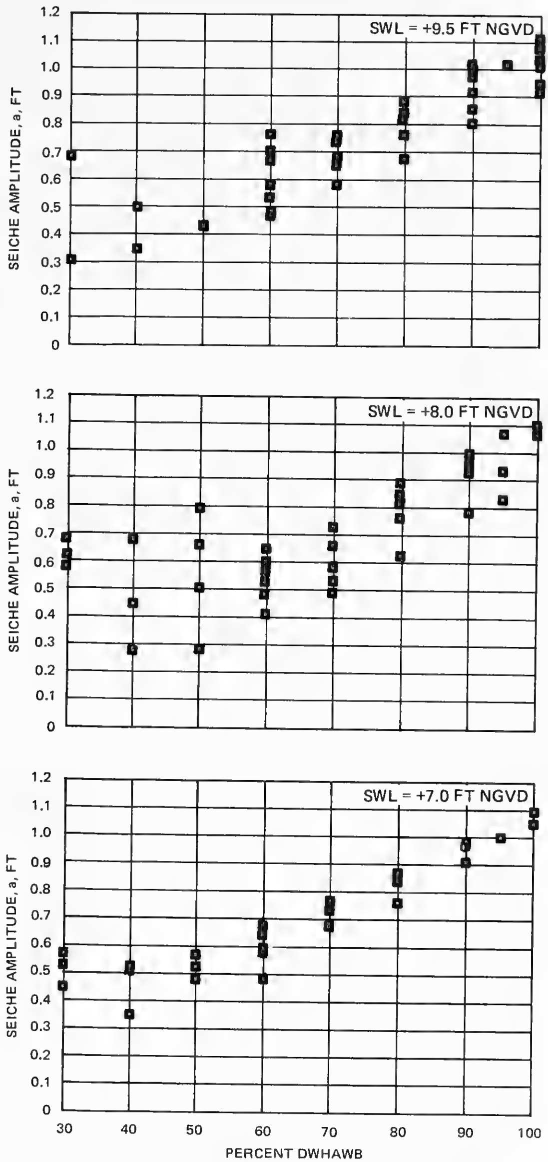


Figure C4. Seiche amplitude versus percent gain (DWHAWB) for Phase II seawall

DWHAWB increased. The seiche wave amplitude was computed by first calculating the H_{mo} associated with seiche. This $\left(H_{mo}\right)_{seiche}$ was calculated by taking the total H_{mo} measured in the tank (single Gage 4) and then subtracting from it the wind wave H_{mo} measured on Goda Array 2 (Gages 5, 6, and 7). Using this $\left(H_{mo}\right)_{seiche}$, the seiche amplitude a_{seiche} was then computed:

$$\left(H_{mo}\right)^2_{total} - \left(H_{mo}\right)^2_{wind} = \left(H_{mo}\right)^2_{seiche} \tag{C1}$$

$$a_{seiche} = \frac{\left(H_{mo}\right)_{seiche}}{4} \sqrt{2} \tag{C2}$$

Some of the seiche in the tank now can be accounted for by changing the form of Equation 3 (main text) to the following:

$$Q' = \exp\left[C_1 F' + \frac{a}{F}\left(C_3 F' - C_2\right)\right] \tag{C3}$$

where

Q' = dimensionless overtopping rate made dimensionless by dividing Q by $\left(gH^3_{mo}\right)^{1/2}$, where g is the acceleration of gravity

C_1, C_2, C_3 = dimensionless regression coefficients

F' = dimensionless relative freeboard parameter

F = average freeboard in feet

a = amplitude of the seiche wave in feet

Figure C5 is a plot of the measured overtopping rate during Phase II seawall tests versus the predicted overtopping rate using Equation C3. Figure C5 shows that the predicted rates agreed well with model results. Figure C6 shows the physical model data again with the predicted Q values from Equation C3 represented by darkened circles. Although the predicted Q values of Figure C6 do not correspond exactly to the measured data values, the figure indicates that much of the data scatter is attributed to seiches in the wave tank. Figure C6 should give the reader confidence in the data collected and should calm any fears created by the data scatter.

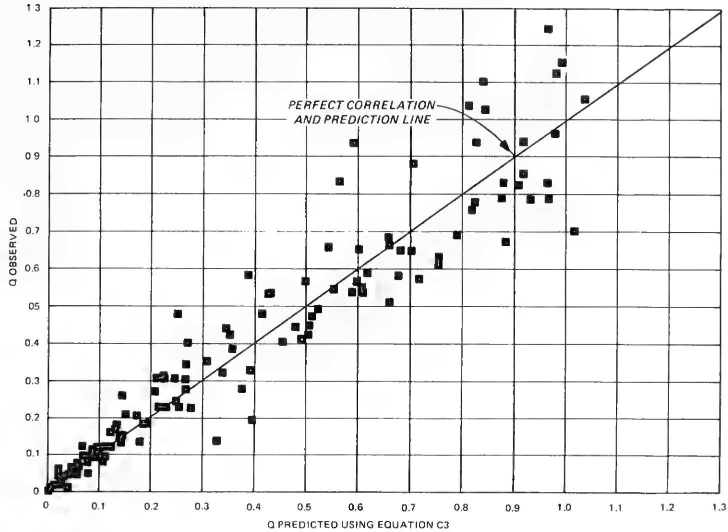


Figure C5. Observed Q versus predicted Q for Equation C3 model

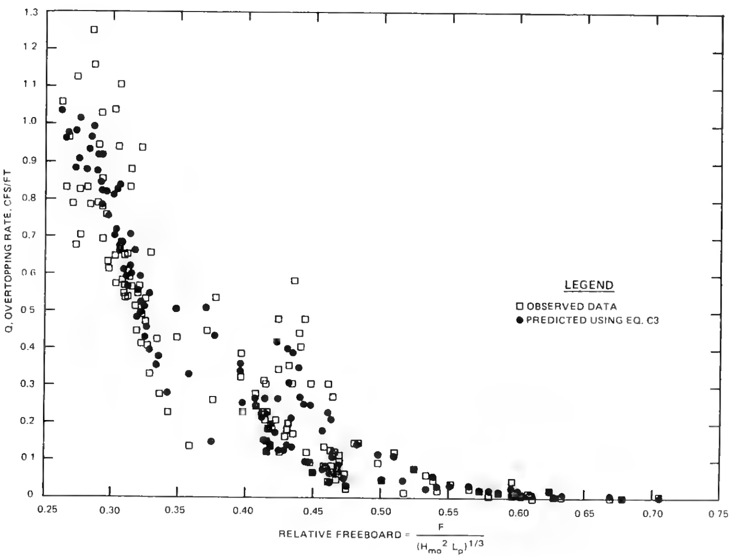


Figure C6. Phase II data plot showing predicted Q values from Equation C3

APPENDIX D: WAVE PRESSURE TEST RESULTS

Presented in Appendix D are two tables, one summarizing the pressure test conditions (Table D1) and the other listing maximum wave-induced pressure (Table D2) to which the seawall was subjected.

Table D1

Summary of Pressure Test Conditions

RUN NO.	STORM	SWL (ft)	GAIN (%)	SAMPLING RATE (Hz)	SAMPLING INTERVAL (sec)
1	HU	7.0	50	2000	300-330
2	HU	7.0	100	2000	300-330
3	HU	7.0	50	2000	600-630
4	HU	7.0	100	2000	600-630
5	HU	7.0	50	2000	900-930
6	HU	7.0	100	2000	900-930
7	HU	7.0	50	2000	1200-1230
8	HU	7.0	100	2000	1200-1230
9	HU	7.0	50	2000	1500-1530
10	HU	7.0	100	2000	1500-1530
11	HU	7.0	50	2000	0-1800
12	HU	7.0	100	80	0-1800
13	HU	8.0	50	80	300-330
14	HU	8.0	100	2000	300-330
15	HU	8.0	50	2000	600-630
16	HU	8.0	100	2000	600-630
17	HU	8.0	50	2000	900-930
18	HU	8.0	100	2000	900-930
19	HU	8.0	50	2000	1200-1230
20	HU	8.0	100	2000	1200-1230
21	HU	8.0	50	2000	1500-1530
22	HU	8.0	100	2000	1500-1530
23	HU	8.0	50	80	0-1800
24	HU	8.0	100	80	0-1800
25	HU	9.5	50	2000	300-330
26	HU	9.5	100	2000	300-330
27	HU	9.5	50	2000	600-630
28	HU	9.5	100	2000	600-630
29	HU	9.5	50	2000	900-930
30	HU	9.5	100	2000	900-930
31	HU	9.5	50	2000	1200-1230
32	HU	9.5	100	2000	1200-1230
33	HU	9.5	50	2000	1500-1530
34	HU	9.5	100	2000	1500-1530
35	HU	9.5	50	80	0-1800
36	HU	9.5	100	80	0-1800
37	HU	9.5	50	1000	300-360
38	NE	9.5	100	1000	300-360
39	NE	9.5	50	1000	600-660
40	NE	9.5	100	1000	600-660
41	NE	9.5	50	1000	900-960
42	NE	9.5	100	1000	900-960
43	NE	9.5	50	1000	1200-1260
44	NE	9.5	100	1000	1200-1260
45	NE	9.5	50	1000	1500-1560
46	NE	9.5	100	1000	1500-1560
47	NE	9.5	50	80	0-1800

(Continued)

(Sheet 1 of 4)

Table D1 (Continued)

RUN NO.	STORM	SWL (ft)	GAIN (%)	SAMPLING RATE (Hz)	SAMPLING INTERVAL (sec)
48	NE	9.5	100	80	0-1800
49	NE	8.0	50	80	0-1800
50	NE	8.0	100	80	0-1800
51	NE	8.0	50	1000	190-250
52	NE	8.0	50	1000	435-495
53	NE	8.0	50	1000	825-885
54	NE	8.0	50	1000	1020-1080
55	NE	8.0	50	1000	1185-1245
56	NE	8.0	50	1000	1710-1770
57	NE	8.0	100	1000	220-280
58	NE	8.0	100	1000	420-480
59	NE	8.0	100	1000	585-645
60	NE	8.0	100	1000	1365-1425
61	NE	8.0	100	1000	1680-1740
62	NE	8.0	100	1000	1740-1800
63	NE	7.0	50	80	0-1800
64	NE	7.0	100	80	0-1800
65	NE	7.0	50	1000	150-210
66	NE	7.0	50	1000	470-530
67	NE	7.0	50	1000	790-850
68	NE	7.0	50	1000	980-1040
69	NE	7.0	50	1000	1265-1325
70	NE	7.0	50	1000	1495-1555
71	NE	7.0	100	1000	120-180
72	NE	7.0	100	1000	180-240
73	NE	7.0	100	1000	340-400
74	NE	7.0	100	1000	420-480
75	NE	7.0	100	1000	540-600
76	NE	7.0	100	1000	600-660
77	NE	7.0	100	1000	765-825
78	HU	7.0	50	80	0-1800
79	HU	7.0	100	80	0-1800
80	HU	7.0	50	1000	285-345
81	HU	7.0	50	1000	445-505
82	HU	7.0	50	1000	510-570
83	HU	7.0	50	1000	600-660
84	HU	7.0	50	1000	690-750
85	HU	7.0	50	1000	875-935
86	HU	7.0	50	1000	1035-1095
87	HU	7.0	50	1000	1275-1335
88	HU	7.0	50	1000	1440-1500
89	HU	7.0	50	1000	1530-1590
90	HU	7.0	50	1000	1605-1665
91	HU	7.0	100	1000	1730-1790
92	HU	7.0	100	1000	1440-1500
93	HU	7.0	100	1000	1380-1440
94	HU	7.0	100	1000	1080-1140
95	HU	7.0	100	1000	1020-1080
96	HU	7.0	100	1000	905-965

(Continued)

(Sheet 2 of 4)

Table D1 (Continued)

RUN NO.	STORM	SWL (ft)	GAIN (%)	SAMPLING RATE (Hz)	SAMPLING INTERVAL (sec)
97	HU	7.0	100	1000	810-870
98	HU	7.0	100	1000	690-750
99	HU	7.0	100	1000	300-360
100	HU	7.0	100	1000	240-300
101	HU	7.0	100	1000	160-220
102	HU	8.0	50	80	0-1800
103	HU	8.0	100	80	0-1800
104	HU	8.0	50	1000	120-180
105	HU	8.0	50	1000	330-390
106	HU	8.0	50	1000	590-650
107	HU	8.0	50	1000	900-960
108	HU	8.0	50	1000	960-1020
109	HU	8.0	50	1000	1080-1140
110	HU	8.0	50	1000	1260-1320
111	HU	8.0	50	1000	1380-1440
112	HU	8.0	50	1000	1620-1680
113	HU	8.0	50	1000	1710-1770
114	HU	8.0	100	1000	60-120
115	HU	8.0	100	1000	130-190
116	HU	8.0	100	1000	200-260
117	HU	8.0	100	1000	270-330
118	HU	8.0	100	1000	330-390
119	HU	8.0	100	1000	400-460
120	HU	8.0	100	1000	460-520
121	HU	8.0	100	1000	530-590
122	HU	8.0	100	1000	600-660
123	HU	8.0	100	1000	675-735
124	HU	8.0	100	1000	780-840
125	HU	8.0	100	1000	850-910
126	HU	8.0	100	1000	1095-1155
127	HU	8.0	100	1000	1180-1240
128	HU	8.0	100	1000	1410-1470
129	HU	8.0	100	1000	1540-1600
130	HU	8.0	100	1000	1660-1720
131	HU	8.0	100	1000	1730-1790
132	HU	9.5	50	1000	1660-1720
133	HU	9.5	50	1000	1730-1790
134	HU	9.5	50	1000	120-180
135	HU	9.5	50	1000	190-250
136	HU	9.5	50	1000	330-390
137	HU	9.5	50	1000	400-460
138	HU	9.5	50	1000	480-540
139	HU	9.5	50	1000	550-610
140	HU	9.5	50	1000	720-780
141	HU	9.5	50	1000	790-850
142	HU	9.5	50	1000	890-950
143	HU	9.5	50	1000	960-1020
144	HU	9.5	50	1000	1130-1190
145	HU	9.5	50	1000	1320-1380

(Continued)

(Sheet 3 of 4)

Table D1 (Concluded)

RUN NO.	STORM	SWL (ft)	GAIN (%)	SAMPLING RATE (Hz)	SAMPLING INTERVAL (sec)
146	HU	9.5	50	1000	1400-1460
147	HU	9.5	50	1000	1470-1530
148	HU	9.5	100	1000	1590-1650
149	HU	9.5	100	1000	1660-1720
150	HU	9.5	100	1000	1730-1790
151	HU	9.5	100	1000	80-140
152	HU	9.5	100	1000	180-240
153	HU	9.5	100	1000	250-310
154	HU	9.5	100	1000	370-430
155	HU	9.5	100	1000	550-610
156	HU	9.5	100	1000	885-945
157	HU	9.5	100	1000	1080-1140
158	HU	9.5	100	1000	1200-1260
159	HU	9.5	100	1000	1320-1380
160	NE	9.5	50	1000	300-360
161	NE	9.5	100	1000	300-360
162	NE	9.5	50	1000	600-660
163	NE	9.5	100	1000	600-660
164	NE	9.5	100	1000	1200-1260
165	NE	9.5	50	1000	1500-1560
166	NE	9.5	100	1000	1500-1560
167	NE	8.0	50	1000	1710-1770
168	NE	8.0	100	1000	220-280
169	NE	8.0	100	1000	420-480
170	NE	8.0	100	1000	585-645
171	NE	8.0	100	1000	1365-1425
172	NE	8.0	100	1000	1680-1740
173	NE	8.0	100	1000	1740-1800
174	NE	8.0	100	1000	1740-1800

(Sheet 4 of 4)

Table D2

Maximum Wave-Induced Pressure

RUN NO.	STORM & SWL (ft)	GAIN (%)	MAXIMUM PRESSURE, PSI, FOR INDICATED ELEVATION,FT. +14.0	+12.5	+10.4	+9.2	+7.7	+6.2
1	H7	50	11.02	4.18	12.73	27.74	10.45	6.08
2	H7	100	25.08	7.22	18.81	16.15	29.64	6.08
3	H7	50	5.89	3.04	15.01	14.63	28.50	27.36
4	H7	100	19.76	5.32	12.54	21.85	34.39	8.55
5	H7	50	6.84	3.23	10.26	20.90	11.40	6.08
6	H7	100	8.17	3.99	9.12	19.38	19.00	17.48
7	H7	50	24.89	4.37	10.83	20.14	31.92	29.07
8	H7	100	4.94	3.80	13.87	9.50	16.15	9.12
9	H7	50	4.37	2.66	17.29	28.50	43.13	16.15
10	H7	100	19.19	9.31	25.08	12.54	26.22	11.02
13	H8	50	13.87	6.84	27.36	16.72	22.99	4.94
14	H8	100	20.14	11.02	13.87	18.81	13.30	8.55
15	H8	50	15.39	7.41	19.95	14.06	19.00	11.02
16	H8	100	11.78	4.94	47.31	13.68	15.96	16.53
17	H8	50	36.29	8.17	16.72	38.57	23.56	15.39
18	H8	100	19.57	8.93	19.19	25.08	18.81	15.58
19	H8	50	16.15	5.70	20.14	74.10	44.08	20.14
20	H8	100	14.44	6.08	31.54	21.09	17.48	5.89
21	H8	50	18.81	9.12	27.17	31.73	34.96	11.78
22	H8	100	24.51	7.22	16.91	13.49	13.30	18.05
25	H95	50	64.22	8.93	20.71	34.96	15.39	4.75
26	H95	100	41.80	10.83	19.57	24.32	11.97	6.46
27	H95	50	58.90	7.22	20.71	24.89	12.73	3.99
28	H95	100	40.28	7.79	44.08	15.77	9.50	11.59
29	H95	50	23.94	6.84	45.98	20.90	20.14	5.32
30	H95	100	35.72	7.60	26.03	16.15	19.57	7.03
31	H95	50	25.46	6.27	39.71	70.49	27.55	3.42
32	H95	100	29.26	6.84	27.36	11.78	11.59	4.56
33	H95	50	42.18	8.74	37.24	22.04	15.96	3.80
34	H95	100	29.83	8.74	12.35	21.47	26.41	5.89
37	NE95	50	43.32	10.45	33.44	17.48	46.17	4.56
38	NE95	100	27.17	9.69	25.08	27.74	19.95	12.73
39	NE95	50	46.55	10.07	27.17	20.52	21.47	4.37
40	NE95	100	41.61	10.26	39.71	22.04	25.84	7.98
41	NE95	50	41.61	9.69	39.14	15.58	17.67	4.37
42	NE95	100	40.09	9.31	30.21	29.64	30.59	7.79
43	NE95	50	53.77	9.12	32.87	43.32	31.92	7.60
44	NE95	100	39.90	10.07	64.98	30.97	24.13	9.31
45	NE95	50	59.47	11.78	65.74	40.66	15.58	10.83
46	NE95	100	53.96	11.97	19.38	22.23	25.65	6.08
51	NE8	50		NEGLIGIBLE				
52	NE8	50	15.39	4.75	24.51	18.43	30.78	11.40
53	NE8	50	26.22	5.32	21.09	18.05	34.96	34.58
54	NE8	50	14.44	5.32	28.12	21.28	33.25	14.06
55	NE8	50	10.07	3.04	30.02	20.90	18.43	12.54
56	NE8	50	19.76	10.64	19.95	23.37	24.32	9.12
57	NE8	100	22.23	10.64	23.37	45.98	22.04	19.76
58	NE8	100	42.18	9.31	15.77	29.07	19.76	10.45

(Continued)

Table D2 (Continued)

RUN NO.	STORM & SWL (ft)	GAIN (%)	MAXIMUM PRESSURE, PSI, FOR INDICATED ELEVATION,FT. +14.0	+12.5	+10.4	+9.2	+7.7	+6.2
59	NE8	100	42.56	11.59	30.78	27.74	15.39	7.03
60	NE8	100	105.64	12.35	30.02	28.69	30.21	9.12
61	NE8	100	36.10	10.64	13.87	22.99	23.37	6.27
62	NE8	100	116.85	13.11	31.92	35.53	13.68	20.14
65	NE7	50	11.21	4.18	29.45	39.14	19.57	10.64
66	NE7	50	11.97	3.42	21.28	19.76	28.69	11.02
67	NE7	50	8.74	4.94	11.59	43.51	28.12	15.20
68	NE7	50	11.97	3.42	23.18	21.66	60.23	21.47
69	NE7	50	12.54	2.28	18.62	21.66	23.75	10.26
70	NE7	50	11.02	3.99	16.72	27.17	19.76	14.63
71	NE7	100	28.12	8.55	15.01	19.95	15.58	11.59
72	NE7	100	19.38	9.50	13.87	29.45	21.66	20.14
73	NE7	100	20.52	8.55	16.72	35.15	15.58	14.82
74	NE7	100	14.63	7.03	54.34	11.97	32.49	14.25
75	NE7	100	10.64	5.89	28.31	27.36	23.18	40.28
76	NE7	100	23.94	8.74	15.96	16.53	21.47	13.11
77	NE7	100	24.32	7.60	10.64	18.24	18.62	19.19
80	H7	50	10.07	3.61	40.85	17.48	25.08	45.03
81	H7	50	8.55	5.89	12.73	22.99	47.50	18.62
82	H7	50	9.88	3.61	19.76	43.32	21.09	28.88
83	H7	50	11.59	4.94	17.86	23.56	17.10	30.97
84	H7	50	25.46	5.89	11.97	27.36	30.02	25.27
85	H7	50	9.88	3.42	35.72	14.44	19.19	88.73
86	H7	50	6.65	2.28	8.36	11.02	17.48	31.35
87	H7	50	5.51	2.28	10.07	13.49	15.01	29.26
88	H7	50	17.48	4.56	15.20	55.67	23.75	19.38
89	H7	50	24.70	8.36	21.66	27.55	41.04	16.15
90	H7	50	10.64	5.70	39.90	15.77	17.86	19.19
91	H7	100	34.01	9.12	22.04	19.95	29.83	9.50
92	H7	100	45.22	10.64	17.10	18.05	27.17	11.21
93	H7	100	56.24	8.74	28.12	29.26	13.30	20.90
94	H7	100	24.70	10.83	18.62	22.23	35.91	13.68
95	H7	100	18.24	6.84	20.33	52.06	26.22	10.26
96	H7	100	31.92	10.45	19.19	19.38	26.22	21.85
97	H7	100	23.18	9.12	25.08	23.18	30.78	18.62
98	H7	100	25.84	9.50	14.63	28.88	13.87	31.92
99	H7	100	40.85	11.78	19.76	19.19	17.67	10.26
100	H7	100	28.12	7.98	15.58	17.48	19.19	11.40
101	H7	100	53.39	9.88	28.12	22.04	15.01	22.61
104	H8	50		NEGLIGIBLE				
105	H8	50		NEGLIGIBLE				
106	H8	50		NEGLIGIBLE				
107	H8	50		NEGLIGIBLE				
108	H8	50		NEGLIGIBLE				
109	H8	50		NEGLIGIBLE				
110	H8	50		NEGLIGIBLE				
111	H8	50		NEGLIGIBLE				
112	H8	50	12.92	4.75	11.97	19.38	75.43	13.87
113	H8	50	16.53	6.27	55.10	16.53	19.19	38.57

(Continued)

(Sheet 2 of 4)

Table D2 (Continued)

RUN NO.	STORM & SWL (ft)	GAIN (%)	MAXIMUM PRESSURE, PSI, FOR INDICATED ELEVATION,FT. +14.0	+12.5	+10.4	+9.2	+7.7	+6.2
114	H8	100	15.39	6.65	15.20	26.22	13.30	21.66
115	H8	100	30.59	7.22	31.35	31.16	21.09	9.31
116	H8	100	26.41	7.98	35.53	22.99	15.77	7.41
117	H8	100	24.32	10.07	21.47	20.14	19.95	23.94
118	H8	100	19.38	7.79	37.24	25.84	21.66	14.82
119	H8	100	35.53	8.74	14.44	18.24	39.52	5.89
120	H8	100	59.66	10.26	20.14	17.10	20.14	17.67
121	H8	100	39.71	10.64	22.23	20.90	63.84	15.96
122	H8	100	68.21	8.93	20.33	13.30	69.16	28.69
123	H8	100	26.03	7.60	72.58	12.54	39.90	6.65
124	H8	100	29.26	7.03	25.27	20.71	17.67	14.82
125	H8	100	16.53	7.22	20.52	24.13	61.94	12.54
126	H8	100	112.67	10.07	31.35	17.10	24.70	8.55
127	H8	100	28.69	7.60	18.05	33.44	24.89	30.21
128	H8	100	63.08	9.12	24.89	21.09	16.53	10.64
129	H8	100	43.51	9.31	25.46	22.80	14.82	11.21
130	H8	100	24.89	7.03	17.86	9.88	14.82	13.49
131	H8	100	19.76	6.27	15.58	19.57	16.15	12.35
132	H95	50	25.65	7.79	55.67	67.45	30.78	12.16
133	H95	50	25.08	8.17	26.98	34.20	16.34	12.92
134	H95	50	48.26	8.93	21.28	19.76	18.43	4.94
135	H95	50	24.89	9.50	78.47	34.39	17.48	4.56
136	H95	50	46.55	10.07	34.77	16.34	34.20	4.75
137	H95	50	28.88	10.07	33.06	23.18	26.79	5.51
138	H95	50	25.65	9.88	50.54	33.06	30.59	6.65
139	H95	50	34.20	7.79	116.85	19.38	26.22	3.42
140	H95	50	56.43	8.55	16.15	29.83	23.18	6.65
141	H95	50	47.12	8.36	38.76	17.48	23.94	16.34
142	H95	50	75.43	10.45	56.05	30.02	34.39	7.22
143	H95	50	29.26	8.93	36.67	30.97	13.87	6.27
144	H95	50	54.53	9.12	27.93	18.43	26.60	4.18
145	H95	50	44.27	10.26	20.14	20.71	22.23	6.65
146	H95	50	31.16	10.07	37.43	22.99	13.87	3.99
147	H95	50	106.02	10.45	22.04	26.79	15.96	9.88
148	H95	100	27.36	9.12	22.42	16.15	19.95	19.57
149	H95	100	19.38	7.03	17.86	17.67	51.87	7.60
150	H95	100	61.37	11.02	27.17	17.86	34.01	9.69
151	H95	100	55.10	8.36	15.20	20.52	22.04	5.51
152	H95	100	60.04	10.64	20.14	19.57	29.45	6.65
153	H95	100	26.98	8.74	20.33	20.14	16.53	9.12
154	H95	100	37.81	9.88	43.32	21.09	17.86	6.65
155	H95	100	33.44	7.98	59.47	11.21	30.78	6.46
156	H95	100	50.54	11.02	24.51	19.57	17.48	6.27
157	H95	100	27.36	9.50	103.17	27.17	28.12	6.46
158	H95	100	29.07	7.79	23.37	13.11	14.63	4.56
159	H95	100	51.11	11.78	30.97	30.78	34.58	15.96
160	NE95	50	45.41	11.21	39.52	23.37	19.00	8.17
161	NE95	100	25.84	10.26	22.61	19.95	30.59	4.37
162	NE95	50	32.11	9.12	20.33	16.34	20.14	4.94

(Continued)

(Sheet 3 of 4)

Table D2 (Concluded)

RUN NO.	STORM & SWL (ft)	GAIN (%)	MAXIMUM PRESSURE, PSI, FOR INDICATED ELEVATION,FT. +14.0	+12.5	+10.4	+9.2	+7.7	+6.2
163	NE95	100	54.72	9.12	34.39	24.13	68.40	8.93
164	NE95	100	46.74	8.74	20.52	11.59	11.59	5.89
165	NE95	50	28.88	9.69	20.52	19.19	34.58	4.56
166	NE95	100	56.05	9.69	38.38	14.06	12.92	7.98
167	NE8	50	19.38	6.65	21.09	84.36	15.20	6.84
168	NE8	100	24.51	6.65	15.20	18.43	22.80	7.22
169	NE8	100	17.48	6.08	19.38	23.75	34.58	6.46
170	NE8	100	33.44	9.31	14.82	40.85	23.18	6.65
171	NE8	100	63.65	10.64	24.32	17.48	20.33	32.11
172	NE8	100	31.54	8.36	14.63	16.53	16.72	6.46
173	NE8	100	42.56	10.07	19.76	19.00	14.44	13.87

(Sheet 4 of 4)

APPENDIX E: NOTATION

B	Value ranging from 0.55 to 0.65
C_1	Dimensionless regression coefficient
d_s	Water depth at the structure toe, ft
F	Average freeboard, or that distance between the crest of the seawall and the local mean water level, ft
F'	Dimensionless relative freeboard
h	Water depth, ft
H_{mo}	Energy based zero-moment wave height, ft
L_p	Significant wave length associated with the peak period T_p, ft
$\frac{L_m}{L_p}$	Linear scale of the model
m,p	Model and prototype quantities, respectively
Q	Overtopping rate, cfs/ft
Q_o	Regression coefficient, cfs/ft
S_a	Specific gravity of an individual stone relative to the water in which it was placed, i.e. $S_a = \gamma_a/\gamma_w$
T_p	Peak period, sec
W_a	Weight of an individual stone, lb
γ	Spectral shape parameter, peak enhancement factor which controls the sharpness of the spectral peak
γ_a	Specific weight of an individual stone, pcf
γ_w	Specific weight of water, pcf
σ_{lo}	Spectral shape parameter, low side decay factor
σ_{hi}	Spectral shape parameter, high side decay factor